STANDARD SPECIFICATIONS AND CODE OF PRACTICE FOR ROAD BRIDGES

SECTION V

STEEL ROAD BRIDGES

(Second Revision)

THE INDIAN ROADS CONGRESS

2001

STANDARD SPECIFICATIONS
AND
CODE OF PRACTICE
FOR
ROAD BRIDGES

Section V
Steel Road Bridges

(Second Revision)

Published by
THE INDIAN ROADS CONGRESS
Jamnagar House, Shahjahan Road
New Delhi-110011
2001

Price Rs.200/
(Plus packing & postage)

First published	: May, 1967
Reprinted	: August, 1972
Reprinted	: July, 1976
Reprinted	: August, 1984 (Incorporates Amendment No.1- December, 1982)
Reprinted	: October, 1994
Second Revision	: April, 2001

Printed at Nutan Printers, New Delhi
(1000 copies)

CONTENTS

Section-V

STEEL ROAD BRIDGES

		Page
	Personnel of Bridges Specifications and Standards Committee	(i) to (iv)
	Introduction	1
501.	Scope	3
502.	Limitations	3
503.	References	4
504.	Definitions and Symbols	4
504.1.	Definitions	4
504.2.	Symbols	5
505.	Materials and Properties	9
505.1.	Steels	9
505.2.	Castings and Forgings	10
505.3.	Fasteners	11
505.4.	Welding Consumables	12
505.5.	Welding	12
505.6.	Wire Ropes and Cables	13
506.	Loads and Stresses	14
506.1.	Combinations	14
506.2.	Permissible Increase in Stress	14
506.3.	Worst Effect	15
506.4.	Working Stresses	15
506.5.	Permissible Stresses in Bolts, Rivets and Tension Rods	16
506.6.	Permissible Stresses in Welds	18
506.7.	Stress Analysis	20

		Page
506.8.	Stresses	21
507.	General Design Considerations	23
507.1.	Effective Spans	23
507.2.	Effective Depths	23
507.3.	Spacing of Girders	23
507.4.	Depth of Girders	23
507.5.	Deflection of Girders	24
507.6.	Camber	25
507.7.	Minimum Sections	25
507.8.	Sectional Area	26
507.9.	Skew Bridges	29
508.	Design of Beams	30
508.1.	General	30
508.2.	Web Plates	30
508.3.	Flanges	32
508.4.	Effective Length of Compression Flanges	33
508.5.	Slenderness Ratio	38
508.6.	Permissible Bending Stresses	38
508.7.	Permissible Shear Stress	45
508.8.	Curtailment of Flange Plates	49
508.9.	Connection of Flanges to Web	49
508.10.	Dispersion of Load Through Flange to Web	49
508.11.	Web Stiffeners	49
508.12.	Flange Splices	55
508.13.	Splices in Webs	56
508.14.	End Connections	56
508.15.	Lateral Bracing	56

		Page
508.16.	Expansion and Contraction	56
509.	Design of Compression Members	57
509.1.	General	57
509.2.	Base Plate	57
509.3.	Cap Plate	61
510.	Design of Tension Members	62
511.	Design of Trusses or Open Web Girders	62
511.1.	General	62
511.2.	Intersection at Joints	62
511.3.	Compression Members	63
511.4.	Tension Members	79
511.5.	Splices	83
511.6.	Connections at Intersection	84
511.7.	Lug Angles	85
511.8.	Section at Pin Holes in Tension Members	85
511.9.	Pin Plates	86
511.10.	Diaphragms in Members	86
511.11.	Lateral Bracings	86
512.	Connections	88
512.1.	Composite Connections	88
512.2.	Welded Connections	88
512.3.	Connections (other than welded)	92
513.	Fabrication and Inspection	98
513.1.	General	98

		Page
513.2.	Laminations in Plates	98
513.3.	Storage of Materials	98
513.4.	Straightening, Bending and Pressing	99
513.5.	Workmanship	100
513.6.	Inspection and Testing	112
514.	Handling, Transportation and Erection	122
514.1.	Scope	122
514.2.	Transportation and Handling	122
514.3.	Storage	123
514.4.	Handling	123
514.5.	Erection Scheme	124
514.6.	Procedure of Erection	124

Appendices

A:	Limitations	127
B:	Rules for Cambering Pre-Deformed Open Web Girder Spans	129
C:	Protection Against Corrosion	133
D:	Post-Construction Inspection and Preventive Maintenance Guidelines	146

PERSONNEL OF BRIDGES SPECIFICATIONS AND STANDARDS COMMITTEE

(As on 24.10.98)

1. Prafulla Kumar (Convenor)	Director General (Road Development), Ministry of Road Transport & Highways, Transport Bhawan, New Delhi-110001
2. The Chief Engineer (B) S&R (Member-Secretary)	(Krishan Kant), Ministry of Road Transport & Highways, Transport Bhawan, New Delhi-110001

Members

3. S.S. Chakraborty	Managing Director, Consulting Engg. Services (I) Ltd., 57, Nehru Place, New Delhi-110019
4. Dr. M.G. Tamhankar	Emeritus Scientist, SERC, 399, Pocket E, Mayur Vihar, Phase-II, Delhi-110091
5. Ninan Koshi	DG (RD) & Addl. Secretary (Retd.), 56, Nalanda Apartments, Vikaspuri, New Delhi-110018
6. A.G. Borkar	Technical Adviser to Metropolitan Commr., A-1, Susnehil Plot No.22, Arun Kumar Vaidya Nagar, Bandra Reclamation, Mumbai-400050
7. N.K. Sinha	Member (Tech.) National Highways Authority of India, 1, Eastern Avenue, Maharani Bagh, New Delhi-110065
8. M.V.B. Rao	Head, Bridges Division, Central Road Research Institute, P.O. CRRI, New Delhi-110020

* ADG(B) being not in position. The meeting was presided by Shri Prafulla Kumar, Director General (Road Development), Ministry of Surface Transport

9. C.R. Alimchandani	Chairman & Managing Director, STUP Consultants Ltd., 1004-5, Reheja Chambers, 213, Nariman Point, Mumbai-400021
10. Dr. S.K. Thakkar	Professor, Department of Earthquake Engg. University of Roorkee, Roorkee-247667
11. M.K. Bhagwagar	Consulting Engineer, Engg. Consultants (P) Ltd., F-14/15, Connaught Place, New Delhi-110001
12. P.D. Wani	Member, Maharashtra Public Service Commission, Bank of India Building, Mumbai-400025
13. S.A. Reddi	Dy. Managing Director, Gammon India Ltd., Gammon House, Prabhadevi, Mumbai-400025
14. Vijay Kumar	Managing Director, UP State Bridge Corporation Ltd., Lucknow-226001
15. C.V. Kand	Consultant; E-2/136, Mahavir Nagar Bhopal-462016
16. M.K. Mukherjee	40/182, C.R. Park, New Delhi-110019
17. Mahesh Tandon	Managing Director, Tandon Consultants (P) Ltd., 17, Link Road, Jangpura Extn. New Delhi-110014
18. Dr. T.N. Subba Rao	Chairman, Construma Consultancy (P) Ltd., 2nd Floor, Pinky Plaza, Mumbai-400052
19. A.K. Harit	Executive Director (B&S), Research Designs & Standards Organisation, Lucknow-226011
20. A.D. Narain	DG (RD) & Addl. Secy., MOST (Retd.), B-186, Sector-26, NOIDA-201301

21. V.R. Jayadas	Chief Engineer, Dy, Director General (B), Hqr. DGBR, Seema Sadak Bhawan, Ring Road, Naraina, New Delhi-110010
22. P.C. Bhasin	324, Mandakini Envlave, Alkananda, New Delhi-110019
23. P.K. Sarmah	Chief Engineer, PWD (Roads) Assam, P.O. Chandmari, Guwahati-781003
24. S.R. Tambe	Secretary, Maharashtra PWD (Retd.), 72, Pranit J. Patkar Marg, Opp. Podar Hospital, Worli, Mumbai-400025
25. The Principal Secretary to the Govt. of Gujarat	(H.P. Jamdar) R&B Department, Block No. 14, New Sachivalaya, 2nd Floor, Gandhinagar-382010
26. The Chief Engineer (R&B)	(D. Sree Rama Murthy), National Highways, Irrum Manzil, Hyderabad-500482
27. The Chief Engineer (NH)	(D. Guha), Public Works Department, Writers' Building, Block-C, Calcutta-700001
28. The Engineer-in-Chief	(K.B. Lal Singhal) Haryana P.W.D., B&R, Sector-19 B, Chandigarh-160019
29. The Director General (Works)	Central Public Works Department, Nirman Bhawan, New Delhi-110001
30. The Chief Engineer	Ministry of Road Transport & Highways, Transport Bhawan, New Delhi-110001
31. The Chief Engineer (R) S&R	(C.C. Bhattacharya), Ministry of Road Transport & Highways, Transport Bhawan, New Delhi-110001
32. The Director	Highways Research Station, 76, Sardar Patel Road, Chennai-600025
33. The Director & Head	Bureau of Indian Standards, Manak Bhawan, 9, Bahadurshah Zafar Marg, New Delhi-110002

34. The Chief Engineer (NH)	M.P. Public Works Department, Bhopal-461004
35. The Chief Engineer (NH)	U.P. P.W.D., Lucknow-226001
36. The Chief Engineer (NH)	Punjab PWD, B&R Branch, Patiala

Ex-Officio Members

37. President, Indian Roads Congress	Maj. C.R. Ramesh, Secretary to the Govt. of Karnataka P.W.D., Bangalore
38. Director General, (Road Development)	Prafulla Kumar, DG (RD) & Addl. Secy. Ministry of Road Transport & Highways, Transport Bhawan, New Delhi-110001
39. Secretary, Indian Roads Congress	S.C. Sharma, Chief Engineer, Ministry of Road Transport & Highways, Transport Bhawan, New Delhi-110001

Corresponding Members

1. N.V. Merani	Principal Secretary (Retd.), A-47/1344, Adarsh Nagar, Worli, Mumbai-400025
2. Dr. G. P. Saha	Chief Engineer, Hindustan Construction Co. Ltd., Hincon House, Lal Bahadur Shastri Marg, Vikhroli (W), Mumbai-400083
3. Shitala Sharan	Advisor Consultant, Consulting Engg. Services (I) Ltd., 57, Nehru Place, New Delhi-110019

INTRODUCTION

The Standard Specifications and Code of Practice for Road Bridges, Section V-Steel Road Bridges, IRC:24-1967 was published by the Indian Roads Congress in 1967. Since this code was brought out more than three decades ago, its revision and updation, commensurate with the current data and incorporation of new concepts and materials has been a long felt need. The work of revision of this code was accordingly taken up by the Steel Bridges Committee (B-7) during its tenure from 1994. The draft was discussed at length during various meetings and finalised. After detailed discussion, the Committee constituted in 1997, consisting the following personnel considered and approved the draft in its meeting held on 21.7.98 for being placed before the Bridges Specifications & Standards Committee.

Vijay Kumar	..	Convenor
R.P. Indoria	..	Member-Secretary

Members

Achyut Ghosh	S.J. Worlikar
M.K. Aggarwal	R.Ray Chaudhuri
Vinod Kumar	K.M. Loomba
T.B. Banerjee	J.N. Pujni
D.D. Sharma	Utpal K. Ghosh
Dr. P. Jayakrishnan	S.Ahmed
A. Ghoshal	A.S. Prasada Rao
H.D. Matange	Prof. Sushil K. Agarwala

Dr. Harshavardhan Subba Rao

Ex-Officio Members

The draft approved by Steel Bridges Committee was discussed by the Bridges Specifications and Standards (BSS) Committee in their meeting on 24.10.98 and approved the same subject to certain modifications. The modified copy approved by Convenor, BSS Committee on 21.6.99 and later it was approved by the Executive Committee on 16.7.99. Finally, the draft was approved by the Council of the Indian Roads Congress at the 156th Council Meeting held at Jaipur on the 6th August, 1999 subject to modifications in light of the comments of Council members. The Convenor, Steel Bridges Committee sent the modified document on 2.6.2000 for forwarding the same to Convenor, BSS Committee for its approval. The Convenor, BSS Committee approved the document for printing on 31.10.2000.

The object of issuing the Standard Specifications and Code of Practice for Steel Road Bridges is to establish a common procedure for the design and construction of road bridges in India.

This revised publication is meant to serve as a guide to both the Design and the Construction Engineer, but compliance with the rules therein does not relieve them in any way of their responsibility for the stability and soundness of the structures designed and erected by them.

The design and construction of road bridges require extensive and thorough knowledge of the science and technique involved and should be entrusted only to specially qualified engineers with adequate practical experience in bridge engineering and capable of ensuring careful execution of work.

501. SCOPE

501.1. This code deals mainly with the design of superstructure of structural steelwork in road bridges. Wherever the provisions of this code do not cover the design requirements in certain particular cases, special literature may be referred to.

501.2. Provisions of this code generally apply to riveted, bolted and welded constructions using hot rolled steel sections only. Cold formed sections are not covered in this code.

501.3. IRC:22-1986 (Section VI) may be referred, wherever applicable in case of concrete work composite with steel.

502. LIMITATIONS

This code generally applies to normal steel bridges. For the following types of bridges for which there are special requirements for design, the provision of the present code will not form adequate basis. This code applies to such bridges to the extent where the special code covering these areas refers to the provisions of this present code. Further reference may be made to ***Appendix-A***.

(a) curved bridges
(b) cable-stayed bridges
(c) suspension bridges
(d) temporary bridges
(e) pedestrian bridges
(f) swing bridges
(g) bascule bridges
(h) box girder bridges
(i) prestressed steel bridges

503. REFERENCES

While preparing this code, practices prevailing in this country in the design and construction of steel bridges have been primarily kept in view. However, recommendations offered in the following publications have also been considered:

(a) IRS Code of Practice for the design of steel or wrought iron bridges carrying rail, road or pedestrian traffic incorporating latest addendum/corrigendum.

(b) BS:5400-Part 3:1982: Code of Practice for Design of Steel Bridges.

504. DEFINITIONS AND SYMBOLS

504.1. **Definitions:** For the purpose of this code, the following definitions shall apply:

504.1.1. **Buckling load:** The load at which a member or a structure as a whole collapses in service or buckles in a load test.

504.1.2. **Dead loads:** The self weights of all permanent construction and installations.

504.1.3. **Effective lateral restraint:** Restraint which produces sufficient resistance in a plane perpendicular to the plane of bending to restrain the compression flange of a loaded strut, beam or girder from buckling to either side at the point of application of the restraint.

504.1.4. **Elastic critical moment:** The elastic moment which will initiate yielding or cause buckling.

504.1.5. **Factor of safety:** The factor by which the yield stress of the material of a member is divided to arrive at the permissible stress in the material.

504.1.6. **Gauge:** The transverse spacing between parallel adjacent lines of fasteners.

504.1.7. **Imposed (live) load:** The load assumed to be produced by the intended use of occupancy including distributed, concentrated, impact, vibration and snow loads but excluding wind and earthquake loads.

504.1.8. **Load factor:** The numerical factor by which the working load is to be multiplied to obtain an appropriate design ultimate load.

504.1.9. **Main member:** A structural member which is primarily responsible for carrying and distributing the applied load.

504.1.10. **Pitch:** The centre to centre distance between individual fasteners in a line of fasteners.

504.1.11. **Secondary member:** Secondary member is that which is provided for stability and or restraining the main members from buckling or similar modes of failure.

504.1.12. **Welding terms:** Unless otherwise defined in this standard the welding terms used shall have the meaning given in IS:812-1957.

504.1.13. **Yield stress:** The minimum yield stress of the material in tension as specified in relevant Indian Standards.

504.1.14. **Warping stress:** Stresses in a box girder due to transverse bending of walls of the box and torsional and distortional warping.

504.2. **Symbols**

Symbols used in this code shall have the following meanings with respect to the structure or member or condition, unless otherwise defined elsewhere in this code :

A	=	Cross-sectional area A (used with subscripts has been defined at appropriate places)
a, b	=	Respectively the greater and lesser projection of the plate beyond column
B	=	Length of side of cap or base
b_0	=	Width of steel flange in encased member
C_m	=	Coefficient
C	=	The distance between centre to centre of battens
c	=	Distance between vertical stiffeners
c_1, c_2	=	Respectively the lesser and greater distances from the neutral axis of the section to the extreme fibres
D	=	Overall depth of beam
d	=	Depth of girder - to be taken as the clear distance between flange angles or where there is no flange angles the clear distance between flanges ignoring fillets.
d_1	= i)	For the web of a beam without horizontal stiffeners : the clear distance between the flanges, neglecting fillets or the clear distance between the inner toes of the flange angles as appropriate.
	= ii)	For the web of a beam with horizontal stiffeners : the clear distance between the horizontal stiffener and the tension flange, neglecting fillets or the inner toes of the tension flange angles as appropriate.
d_2	=	Twice the clear distance from the neutral axis of a beam to the compression flange, neglecting fillets or the inner toes of the flange angles as appropriate
E	=	The modulus of elasticity for steel
f_{cb}	=	Elastic critical stress in bending
f_{cc}	=	Elastic critical stress in compression, also known as Euler critical stress
f_y	=	Yield stress
g	=	Gauge
h	=	Outstand of the stiffener
I	=	Moment of inertia

k	=	Distance from outer face of flange to web toe of fillet of member to be stiffened
k_1, k_2	=	Coefficients
k_b or k_c	=	Flexural stiffnesses
L	=	Span/length of member
l	=	Effective length of the member
M	=	Bending moment
M_p	=	Maximum moment (plastic) capacity of a section
M_{pc}	=	Maximum moment (plastic) capacity of a section subjected to bending and axial loads
M_u	=	Lateral buckling strength in the absence of axial load
N	=	Number of parallel planes of battens
n	=	Coefficient in the Merchant Rankine formula, assumed as 1.4
P	=	Axial force, compressive or tensile
P_m	=	Calculated maximum load capacity of a strut
P_{ut}	=	Calculated maximum load capacity as a tension member
P_e	=	Euler load
P_y	=	Yield strength of axially loaded section
R	=	The reaction of the beam at the support
r	=	Radius of gyration of the section
S	=	Transverse distance between centroids of rivet groups or welding
s	=	Staggered pitch
T	=	Mean thickness of compression flange (T used with subscripts has been defined at appropriate place)
t	=	Thickness of web
V	=	Transverse shear
V_l	=	Longitudinal shear
V_s	=	Calculated maximum shear capacity of a section
W	=	Total load
w	=	Pressure or loading on the underside of the base
Zp	=	Plastic modules of the section
β	=	Ratio of smaller to larger moment
β_1, β_2	=	Stiffness ratio
λ	=	Coefficient

λ = Slenderness ratio of the member, ratio of the effective length (l) to the appropriate radius of gyration (r)

λ_0 = Characteristic slenderness ratio = $\sqrt{Py/Pe}$

θ = Ratio of the rotation at the hinge point to the relative elastic rotation of the far end of the beam segment containing plastic hinge

σ_{ac} = Maximum permissible compressive stress in an axially loaded strut not subjected to bending

σ_{at} = Maximum permissible tensile stress in an axially loaded tension member not subjected to bending

σ_{bc} = Maximum permissible compressive stress due to bending in a member not subjected to axial force.

σ_{bg} = Maximum permissible bearing stress on flat surface

σ_{bs} = Maximum permissible bending stress in slab base

σ_{bt} = Maximum permissible tensile stress due to bending in a member not subjected to axial force

σ_{cc} = Maximum permissible stress in concrete in compression

σ_{e} = Maximum permissible equivalent stress

σ_{p} = Maximum permissible bearing stress in a member

σ_{pf} = Maximum permissible bearing stress in a fastener

σ_{sc} = Maximum permissible stress in steel in compression

σ_{tf} = Maximum permissible stress in axial tension in fastener

$\sigma_{ac,\,cal}$ = Calculated average axial compressive stresses

$\sigma_{at,\,cal}$ = Calculated average stress in member due to an axial tensile force

$\sigma_{bc,\,cal}$ = Calculated compressive stress in a member due to bending about a principal axis

$\sigma_{bt,\,cal}$ = Calculated tensile stress in a member due to bending about both principal axes

$\sigma_{e,\,cal}$ = Calculated equivalent stress

T_{va} = Maximum permissible average shear stress in member

T_{vm} = Maximum permissible shear stress in a member

T_{vf} = Maximum permissible shear stress in fastener

$\tau_{v, cal}$ = Calculated shear stress in a member

ω = Ratio of moment of inertia of the compression flange alone, to that of the sum of the moments of inertia of the flanges, each calculated about its own axis parallel to the y-y axis of the girder, at the point of maximum bending moment.

γ = Ratio of total area of both the flanges at the point of least bending moment to the corresponding area at the point of greatest bending moment

Note: The subscripts x, y denote the x-x and y-y axis of the section respectively. For symmetrical sections, x-x denotes the major principal axis while y-y denotes the minor principal axis.

505. MATERIALS AND PROPERTIES

505.1. Steels

505.1.1. **Properties of steel:** The following properties shall be assumed for all grades of steel for design purposes:

Young's Modulus (Modulus of Elasticity)	= 2.11×10^5 Mpa
Shear Modulus	= 77×10^3 Mpa
Poisson's Ratio	= 0.30
Coefficient of Thermal Expansion	= 0.0000117/^{0}c/unit length

505.1.2. **Structural steels:** Unless otherwise permitted herein, all structural steel shall, before fabrication comply with the requirements of the following Indian Standards, or their latest revisions as appropriate :-

IS:808-1989	Dimensions for hot rolled steel beam, column, channel and angle sections
IS:1161-1979	Steel tubes for structural purposes
IS:1239 (Pt 1)-1990	Mild steel tubes, tubulars and other wrought steel fittings:Part 1 Mild steel tubes
IS:1239 (Pt 2)-1992	Mild steel tubes, tubulars and other wrought steel fittings : Part 2 Mild steel tubulars and other wrought steel fittings

IS:1730-1989	Dimensions for steel plates, sheets, strips and flats for general engineering purposes
IS:1732-1989	Dimension for round and square steel bars for structural and general engineering purposes
IS:1852-1973	Rolling and cutting tolerances for hot rolled steel products
IS:2062-1992	Steel for general structural purposes
IS:4923-1985	Hollow steel sections for structural use
IS:8500-1992	Structural steel microalloyed (medium and high strength qualities)
IS:11587-1986	Structural weather resistant steels

The use of structural steel not covered by the above standards may be permitted with the specific approval of the authority.

505.1.3. **Other steels:** Except where permitted with the specific approval of the authority, steels for machined parts and for uses in other than structural members or elements shall comply with the following or relevant Indian Standards.

IS: 1875-1992	Carbon steel billets, blooms, slabs and bars for forgings
IS: 6911-1992	Stainless steel plate, sheet and strip

505.2. **Castings and Forgings**

Steel casting and forgings shall comply with the requirements of the following Indian Standards as appropriate :

IS:1030-1989	Carbon steel castings for general engineering purposes
IS:1875-1992	Carbon steel billets, blooms, slabs & bars for forgings
IS:2004-1991	Carbon steel forgings for general engineering purposes

IS:2644-1986	High tensile steel castings
IS:4367-1991	Alloy steel forgings for general industrial use

505.3. **Fasteners**

Bolts, nuts, washers and rivets shall comply with the following or relevant Indian Standards, as appropriate:

IS:1148-1982	Hot rolled rivet bars (upto 40mm dia) for structural purposes
IS:1149-1982	High tensile steel rivet bars for structural purposes
IS:1363-1992 (Pt 1 to Pt 3)	Hexagon head bolts, screws and nuts of product grade C (size range M5 to M64)
IS:1364-1992	Hexagon head bolts,screw and nuts
(Pt 1 to Pt 3)	products grade A & B (size range M1.6 to M64)
IS:1367-1979~94 (Pt 1 to Pt 18)	Technical supply conditions for threaded steel fasteners
IS:1929-1982	Hot forged steel rivets for hot closing (12 to 36 mm diameter)
IS:2155-1982	Cold forged solid steel rivets for hot closing (6 to 16 mm diameter)
IS:3640-1982	Hexagon fit bolts
IS:3757-1985	High strength structural bolts
IS:4000-1992	High strength bolts in steel structures-code of practice
IS:5369-1975	General requirements for plain washers and lock washers
IS:5370-1969	Plain washers with outside dia $\cong$ 3 x inside dia.
IS:5372-1975	Taper washers for channels (ISMC)
IS:5374-1975	Taper washer for I beams (ISMB)
IS:5624-1970	Foundation bolts
IS:6610-1972	Heavy washers for steel structures

IS:6623-1985	High strength structural nuts
IS:6649-1985	Hardened and tempered washers for high strength structural bolts and nuts
IS:7002-1991	Prevailing torque type steel hexagon nuts

505.4. **Welding Consumables**

Welding consumables shall comply with the following Indian Standards, as appropriate :

IS:814-1991	Covered electrodes for manual metal are welding of carbon and carbon manganese steel
IS:1395-1982	Low and medium alloy, steel covered electrodes for manual metal arc welding
IS:3613-1974	Acceptance tests for wire flux combination for submerged arc welding
IS:6419-1971	Welding rods and bare electrodes for gas shielded arc welding of structural steel
IS:6560-1972	Molybdenum and chromium - molybdenum low alloy steel welding rods and bare electrodes for gas shielded arc welding
IS:7280-1974	Bare wire electrodes for submerged arc welding of structural steel

505.5 **Welding**

IS:812-1957	Glossary of terms relating to welding and cutting of metal
IS:816-1969	Code of practice for use of metal arc welding for general construction in mild steel
IS:822-1970	Code of procedure for inspection of welds
IS:1024-1979	Code of practice for use of welding in bridges and structures subject to dynamic loading

IS:1182-1983	Recommended practice for radiographic examination of fusion welded butt joints in steel plates
IS:4853-1982	Recommended practice for radiographic inspection of fusion welded butt joints in steel pipes
IS:5334-1981	Code of practice for magnetic particle flaw detection of welds
IS:7307(Pt. 1) -1974	Approval tests for welding procedures: Part-I fusion welding of steel
IS:7310(Pt. 1) -1974	Approval tests for welders working to approved welding procedures:Part-1 fusion welding of steel
IS:7318(Pt. 1) -1974	Approval tests for welders when welding procedure is not required : Part-1 fusion welding of steel
IS:9595-1980	Recommendations for metal arc welding of carbon and carbon manganese steels

505.6. **Wire Ropes and Cables**

These shall conform to the following or relevant Indian Standards except where use of other types is specifically permitted by the authority.

IS:1785 (Pt. 1) -1983	Specification for plain hard-drawn steel wire for prestressed concrete : Part-1 Cold drawn stress relieved wire
IS:1785 (Pt. 2) -1983	Specification for plain hard-drawn steel wire for prestressed concrete : Part-2 As- drawn wire
IS:2266-1989	Steel wire ropes for general engineeering purposes
IS:2315-1978	Thimbles for wire ropes
IS:9282-1979	Wire ropes and strands for suspension bridges

506. LOADS AND STRESSES

506.1. Combinations

506.1.1. **Main effects:** For the purpose of computing stresses, the classifications (column 1) and combinations (column 2) as given in Table 6.1 below will be followed. For legend of symbols under combination (column 2) refer to Clause 202.1 of IRC:6-2000 (Fourth Revision).

506.1.2. **Other effects**

506.1.2.1. Secondary Effects (F_s) shall include, where applicable, the effects due to creep and shrinkage of concrete for composite deck and warping for box girder sections.

506.1.2.2. Erection effects shall include the loads and forces arising out of construction equipment and the effects of wind/seismic.

506.2. Permissible Increase in Stress

506.2.1. **Increase:** The permissible increase (per cent) in stress in the various members covered by this code due to combination stated in Clause 506.1 shall be as given under Increase (column 3) of Table 6.1.

Table 6.1. Permissible Increase in Stress

Classification (1)	Combination (2)	Increase (3)
I	$G + Q$ or $G_s + Q_{im} + F_{wc} + F_f + G_b + F_{cf} + F_{ep} + G_c$	Nil
II	$(I) + F_s + F_d + F_{te}$	15 per cent
III	$(II) + W + F_{wp}$	25 per cent
IV	$(II) + F_{eq} + F_{wp}$	40 per cent
V	$(II) + F_{im} + W$	25 per cent
VI	$G + F_{wc} + G_b + F_{ep} + F_{er} + F_f + W + G_c$	30 per cent
VII	$(VI) + F_{eq} - W$	40 per cent

506.2.2. **Limitation:** The above permissible increase in stress, shall, however, be limited to 90 per cent of yield stress.

506.3. **Worst Effect**

Subject to the provision of other clauses, all forces shall be considered as applied and all loaded lengths chosen in such a manner that the worst adverse effect is caused on the member under consideration.

506.4. **Working Stresses**

506.4.1 **Basic permissible stresses:** The basic permissible stresses for steel work are given in Table 6.2.

Table 6.2. Basic Permissible Stresses

1.	Axial tension on net area	$0.6 f_y$
2.	Axial compression on effective section	$0.6 f_y$
3.	Bending	
	In plates, flats, tubes and similar sections	$0.66 f_y$
	In girders and rolled sections	$0.62 f_y$
4.	Shear Stress	
	Maximum	$0.43 f_y$
	Average	$0.38 f_y$
	For yield stress $fy < = 250$ Mpa	
	For $Fy > 250$ Mpa	$0.35 f_y$
5.	Bearing stress on flat surface	$0.8 f_y$

However, the permissible stresses in axial or flexural compression shall not exceed those as per relevant clauses considering the effect of buckling.

506.4.2. **Equivalent Stress**

506.4.2.1. When a member is subjected to a combination of stresses, the equivalent stress $\sigma_{e, cal}$ due to combination of shear stress $\tau_{v, cal}$ bearing stress $\sigma_{p, cal}$ and bending stress $\sigma_{bt, cal}$ tensile or $\sigma_{bc, cal}$ compressive is calculated from

$$\sigma_{e.cal} = \sqrt{(\sigma_{bt.cal})^2 + (\sigma_{p.cal})^2 + (\sigma_{bt.cal})(\sigma_{p.cal}) + 3(\tau_{vcal})^2}$$

$$\text{or } \sigma_{e.cal} = \sqrt{(\sigma_{bc.cal})^2 + (\sigma_{p.cal})^2 - (\sigma_{p.cal}) + 3(\tau_{vcal})^2}$$

$\sigma_{cal}\sigma,_{e}$ shall not exceed the permissible stresses as indicated in relevant sections under different combination of stresses.

506.4.2.2. Irrespective of the permissible increase of stress in other clauses. the equivalent stress $\sigma_{e.cal}$ calculated in Clause 506.4.2.1 above shall not exceed 92 per cent of yield stress.

506.5. **Permissible Stresses in Bolts, Rivets and Tension Rods**

506.5.1. **Fasteners:** All fasteners would be in accordance with Indian Standards. For bolts, the yield stress used for calculating the permissible stress would be derived from the property class chosen as per relevant Indian Standards. The nut should be of matching property class. For hot rolled and high tensile rivets, the yield stress would be in accordance with the relevant Indian Standards.

506.5.2. **Calculation of stresses:** In calculating shear and bearing stresses, the effective diameter of a rivet shall be taken as the hole diameter and that of bolt shall be taken as its nominal diameter. In calculating the axial tensile stress in a rivet, the gross area shall be used and in calculating the axial tensile stress in a bolt or screwed tension rod, the net area shall be used.

506.5.3. **Gross and net area**

506.5.3.1. The gross area of a rivet shall be taken as the cross-sectional area of the rivet hole. The nominal diameter of rivet shall be the diameter (cold) before driving. The nominal area of a rivet shall be based on the nominal diameter.

506.5.3.2. The net sectional area of a bolt or a screwed tension rod shall be taken as the area of the root of the threaded part or cross-sectional area of the unthreaded part whichever is lesser. The nominal diameter of a bolt shall be the diameter of the shank of the bolt. The nominal area of a bolt shall be based on the nominal diameter.

506.5.4. **Basic permissible stresses:** The basic permissible stresses for rivets, bolts, tension rods are given in Table 6.3

506.5.5. **Combined tensile and shear stresses:** Rivets and bolts subject to shear and externally applied tensile forces shall be so proportioned that the quantity.

$$[(\sigma_{tf,cal}/\sigma_{tf})^2+(\tau_{vf,cal}/\tau_{vf})^2]\leq 1$$

Where

$\sigma_{tf,cal}$ = actual tensile stress in the rivet or bolt

σ_{tf} = permissible tensile stress in the rivet or bolt as given in Table 6.3

$\tau_{vf,cal}$ = actual shear stress in the rivet or bolt,

τ_{vf} = permissible shear stress in the rivet or bolt as given in Table 6.3

506.5.6. **HSFG bolts :** High strength friction grip (HSFG) bolts shall be used in conformity with IS:4000-1992.

Table 6.3. Basic Permissible Stresses for Rivets, Bolts and Tension Rods

1.	In tension Axial stress on nominal area of rivet and on net area of bolts and tension rods : Power driven shop rivets Power driven field rivets Bolts over 38 mm dia Bolts 20 mm upto 38 mm dia Bolts less than 20 mm dia Tension rods	 $0.33 f_y$ $0.27 f_y$ $0.53 f_y$ $0.40 f_y$ $0.33 f_y$ $0.53 f_y$
2.	In shear Shear stress on gross area of rivets and nominal area of bolts : Power driven shop rivets Power driven field rivets Hand driven rivets Turned and fitted bolts (IS:3640) Black bolts (IS:1363)	 $0.43 f_y$ $0.40 f_y$ $0.33 f_y$ $0.43 f_y$ $0.37 f_y$
3.	In bearing Bearing stress on gross diameter of rivets and nominal diameter bolts: Power driven shop rivets Power driven field rivets Hand driven rivets Turned and fitted bolts (IS:3640) Black bolts (IS:1363)	 $1.00 f_y$ $0.90 f_y$ $0.67 f_y$ $1.00 f_y$ $0.87 f_y$

506.6. Permissible Stresses in Welds

506.6.1. **Basic permissible stresses:** The basic permissible stresses in weld shall be as per Indian Standards namely IS:816-1969 and as modified in IS:1024-1979.

506.6.2. **Shop welds**

506.6.2.1. **Butt welds:** Butt weld shall be treated as parent metal with a thickness equal to the throat thickness, and the stress shall not exceed those permitted in the parent metal.

506.6.2.2. **Fillet welds:** The basic permissible stress in fillet welds shall not exceed the permissible shear stress as follows :

Steel Conforming as per IS:815-1974	Electrode Designation	Shear Stress Mpa
IS:2062-1969	EXXX-43X	108
IS:8500-1991 Grade Fe 510 W-HT	EXXX-51X	131

506.6.2.3. **Plug welds:** The permissible shear stress in plug welds will not exceed those given for fillet welds as above.

506.6.3. **Site welds:** The permissible stresses for shear and tension for site welds made during erection of structural members shall be reduced to 80 per cent of those given in Clause 506.6.2. Site welding should be proposed only if quality welds can be ensured at site including facilities for testing the welds as per codal requirements. The percentage of site welds to be tested should be 100 per cent as given under Clause 513.6.12.7.2 to 4.

506.6.4. **Combined stresses in a weld**

506.6.4.1. When a weld is subjected to a combination of stresses, the equivalent stress $\sigma_{e,cal}$ due to combination of shear stress $\tau_{v,cal}$ bearing stress $\sigma_{p,cal}$ and bending stress $\sigma_{bt,cal}$ tensile or $\sigma_{bc,cal}$ compressive is calculated from

$$\sigma_{e,cal} = \sqrt{(\sigma_{bt,cal})^2+(\sigma_{p,cal})^2+(\sigma_{bt,cal})(\sigma_{p,cal})+3(\tau_{v,cal})^2}$$

$$\text{or } \sigma_{e,cal} = \sqrt{(\sigma_{bc,cal})^2+(\sigma_{p,cal})^2+(\sigma_{bc,cal})(\sigma_{p,cal})+3(\tau_{v,cal})^2}$$

$\sigma_{e,cal}$ shall not exceed the permissible stresses as indicated in relevant sections under different combination of stresses.

506.6.4.2. Irrespective of the permissible increase of stress in other clauses, the equivalent stress $\sigma_{e\,cal}$ calculated in Clause 506.6.4.1 shall not exceed 92 per cent of yield stress f_y.

506.7. **Stress Analysis**

506.7.1. **General:** The global analysis of the structure should be done using an elastic method. For structures in which the load effects are not proportional to the loads and/or the secondary effects due to deformation are significant, the method of analysis should be suitable for treatment of non-linear behaviour.

506.7.2. **Sectional properties:** The sectional properties to be used in global analysis should generally be calculated for the gross section assuming the specified sizes. For beams or trusses on flexible supports account should, however, be taken of its influence of shear lag on their stiffnesses. The effect of shear lag should also be taken into account in analysis of conditions during erection of continuous girders of box construction or with integral decks.

506.7.3. **Longitudinal stresses in beams:** The distribution of longitudinal stress between the flanges and web or webs of a beam may be calculated on the assumption that plane section remains plane, but using effective widths of the flanges and the effective thickness of a deep web in accordance with the provisions of Clause 508, no further account need be taken of deformation of the plate out of its plane.

506.7.4. **Shear stress:** The design values of shear stress in webs of rolled or fabricated I, box or channel sections may be calculated in accordance with the provisions of Clause 508. Shear stresses in other

sections should be computed from the whole cross-section having regard to the distribution of flexural stress across the section.

506.8. **Stresses**

506.8.1.**Primary stresses:** In the design of triangulated structures, axial stresses in members are usually calculated on the assumption that :

- all members are straight and free to rotate at the joints;
- all joints lie at the intersection of the centroidal axes of the members
- all loads, including the weight of the members, are applied at the joints.

These stresses are defined as primary stresses.

506.8.2.**Secondary stresses:** In practice these assumptions are not realised and consequently members are subjected not only to axial stress but also to bending and shear stresses. These stresses are defined as secondary stresses and fall into two groups :

(i) Stresses which are the result of eccentricity of connections and off-joint loading generally (i.e., loads rolling directly on chords, self weight of member and wind loads on member)

(ii) Stresses which are the result of the elastic deformation of the structure and the rigidity of the joints. These are known as deformation stresses.

506.8.2.1. Structures shall be designed, fabricated and erected in such a manner as to minimise as far as possible secondary stresses.

506.8.2.2. Secondary stresses which are the result of eccentricity of connections and of off-joint loading [under Clause 506.8.2 (i)], shall be computed and combined with the co-existent axial stresses in

accordance with appropriate Clause, but secondary stress due to the self weight and wind on the member shall be ignored in this case.

Note : In computing the secondary stress due to loads being carried direct by a chord, the chord may be assumed to be a continuous girder supported at the panel points, the resulting bending moments, both at the centre and at the supports being taken as equal to 3/4 of the maximum bending moment in a simply supported beam of span equal to the panel length. Where desired, calculations may be made and the calculated bending moments may be taken. In computing such bending moments, the impact allowances shall be based on a loaded length equal to one panel length.

506.8.2.3. Secondary stresses which are the result of the elastic deformation of the structure [under Clause 506.8.2 (ii)] shall be either computed or assumed in accordance with Clause 506.8.3 and combined with the co-existent axial stresses.

506.8.3. **Deformation stresses:** In order to minimise the deformation stresses in girder, the ratio of the width of the members in the plane of distortion to their length between centre of intersections shall preferably be not greater than 1/12 of the chord members and 1/24 of web members. In the absence of calculations the deformation stresses shall be assumed to be not less than $16\frac{2}{3}$ per cent of the dead load and live load stresses including impact.

506.8.3.1. All open web girders of effective spans greater than 50 m may properly be cambered. Recommended procedure for cambering such girders is given in ***Appendix-B***. For such girders, deformation stresses (under Clause 506.8.3) may be ignored.

507. GENERAL DESIGN CONSIDERATIONS

507.1. Effective Spans

The effective span shall be as given below :

(a) *For main girders* - The distance between centres of bearings

(b) *For cross girders* - The distance between the centres of the main girders or trusses

(c) *For rail or road bearers* - The distance between the centres of cross members

Note:-Where a cross girder or bearer terminates on an abutment or pier, the centre of bearing thereon shall be taken as one end of the effective span.

(d) *For pins in bending* - The distance between the centres of bearings; but where pins pass through bearing plates having thickness greater than half the diameter of the pins, consideration may be given to the effect of the distribution of bearing pressures on the effective span.

507.2. Effective Depths

The effective depth of plate or truss girder should be taken as the distance between the centres of gravity of the upper and lower flanges or chords.

507.3. Spacing of Girders

The distance between centres between the main girders shall be sufficient to resist over turning or overstressing due to lateral forces and loading conditions. Otherwise special provisions must be made to prevent this. This distance shall not be less than 1/20 of the span.

507.4. Depth of Girders

Minimum depth preferably shall not be less than the following :

(a) For trusses : 1/10 of the effective span

(b) For rolled steel joists and plate girders : 1/25 of the effective span

(c) For composite steel and concrete bridge :

(i) Overall depth : 1/25 of the effective span

(ii) Steel beam or girder : 1/30 of the effective span

The effective depth of open web girders shall not be greater than three times the distance between the centres of main girders.

507.5. **Deflection of Girders**

507.5.1. Rolled steel beams, plate girders or lattice girders, either simple or continuous spans, shall be designed so that the total deflection due to dead load, live load and impact shall not exceed 1/600 of the span.

Additionally, the deflection due to live load and impact shall not exceed of 1/800 of the span.

507.5.2. The deflection of cantilever arms at the tip due to dead load, live load and impact shall not exceed 1/300 of the cantilever arm and deflection due to live load and impact shall not exceed 1/400 of the cantilever arm.

507.5.3. Sidewalk live load may be neglected in calculating deflection.

507.5.4. When cross bracings or diaphragms of sufficient depth and strength are provided between beams to ensure the lateral distribution of loads the deflection may be calculated considering all beams acting together.

507.5.5. The gross moment of inertia shall be used for calculating the deflection of beams or plate girders. In calculating the deflection of trusses, the gross area of each member should be used.

507.6. **Camber**

507.6.1. Camber, if any, shall be provided as specified by the engineer. Camber may be required to maintain clearance under all conditions of loading or it may be required on account of appearance.

507.6.2. In the absence of specific guidance, the following principles may be observed:

(a) Beams and plate girders upto and including 35 m span need not be cambered.

(b) In open web spans, the camber of the main girders and the corresponding variations in length of members shall be such that when the girders are loaded with full dead load plus 75 per cent of the live load without impact producing maximum bending moment, they shall take up the true geometrical shape assumed in their design. The camber diagram shall be prepared as indicated in ***Appendix-B.***

507.7. **Minimum Sections**

507.7.1. For all members of the structure, except parapets and packing plates, the following minimum thicknesses of plates and rolled sections shall apply :

(a) 8 mm when both sides are accessible for painting or are in close contact with other plates or rolled sections, or are otherwise adequately protected against corrosion. When one side is not readily accessible for painting or is not in close contact with another member, or is not otherwise adequately protected and where the thickness required by calculation is less than 12.5 mm, 1.5 mm shall be added to the calculated thickness subject to the total thickness being not less than 10 mm.

(b) 6 mm for box members when the inside of the member is effectively sealed.

(c) For rolled steel beams and channels, the controlling thickness

shall be taken as the mean thickness of the flange, regardless of the web thickness.

507.7.2. In floor plates and parapets, minimum thickness of 6 mm shall be used if both sides are exposed or 8 mm if only one side is exposed. For packing plates, the thickness shall not be less than 1.5 mm.

507.7.3. In riveted construction, no angle less than 75 x 50 mm shall be used for the main members of the girders.

507.7.4. No angle less than 65 x 45 mm and no flat less than 50 mm wide shall be used in any part of a bridge structure, except for hand railings and shear connectors.

507.7.5. End angles connecting stringers to cross girders or cross girders to main girders shall not be less than three quarters of the thickness of the web plates of the stringers and cross girders respectively.

507.8. **Sectional Area**

507.8.1. **Gross sectional area:** The gross sectional area shall be the area of the cross-section as calculated from the specified sizes.

507.8.2. **Effective sectional area**

507.8.2.1. Tension members- The effective sectional area of the member shall be the gross sectional area with the following deductions as appropriate.

507.8.2.1.1. Except as required in Clause 507.8.2.1.2 the areas to be deducted shall be the sum of the sectional areas of the maximum number of holes in any cross-section at right angles to the direction of stress in the member.

507.8.2.1.2. In the case of :

(a) all axially loaded tension members

(b) beams of structural steel conforming to IS : 2062 and with d/t greater than 85

(c) beams of structural steel conforming to IS:8500 and with d/t greater than 75

where,

t = thickness of web, and

d = depth of beams to be taken as the clear distance between flanges ignoring fillets.

and where bolt or rivet holes are staggered, the area to be deducted shall be the greater of the following :

(i) the maximum number of the holes in any cross-section at right angles to the direction of stress in the member.

(ii) the sum of the sectional areas of all holes in a chain of lines extending progressively across the member, less $s^2 t/4g$ for each line extending between holes at other than right angles to the direction of stress, where, s, g and t are respectively the staggered pitch, gauge, and thickness associated with the line under consideration. The chain of lines shall be chosen to produce the maximum such deduction. For non-planer sections, such as, angles, with holes in both legs, the gauge, g, shall be the distance along the centre of the thickness of the section between hole centres.

Note : In a built-up member where the chains of holes considered in individual parts do not correspond with the critical chain of holes for the members as a whole, the value of any rivets or bolts joining the parts between such chains of holes shall be taken into account in determining the strength of the member.

507.8.2.1.3. *Angles and tees in tension*

(a) In the case of single angle connected through one leg the net effective sectional area shall be taken as :

$$A1 + A2 \times k$$

where,

$A1$ = effective cross-sectional area of the connected leg

$A2$ = the gross cross-sectional area of the unconnected leg, and

$$k = 3A1 \div (3A1 + A2)$$

Where lug angles are used, the effective sectional area of the whole of the angle member shall be considered.

(b) In the case of pair of angles back-to-back (or a single tee) connected by one leg of the angle (or by the flange of the tee) to the same side of a gusset, the net effective area shall be taken as

$$A1 + A2 \times k$$

where,

$A1$ and $A2$ are as defined in Clause 507.8.2.1.3 (a) and

$$k = 5A1 \div (5A1 + A2)$$

The angles shall be connected together along their length in accordance with the requirements as given in Clause 511.4.6.1.

507.8.2.2. **Compression members:** The gross sectional area shall be taken for all compression members subject to relevant clauses.

507.8.2.3. **Parts in shear:** The effective sectional area for calculating average shear stress for parts in shear shall be as follows :

(a) *Rolled beams and channels* - The product of the thickness of the web and the overall depth of the section.

(b) *Plate girder* - The product of the thickness of the web and the full depth of the web plate.

Note:- (1) Where webs are varied in thickness in the depth of the section by the use of tongue plates or the like and in the case of other sections, the maximum shear stress shall be computed from the whole area of cross-section having regard to the distribution of flexural stresses.

(2) Webs which have openings larger than those used for rivets, bolts or other fastenings require special consideration and the provisions of this clause are not applicable.

507.9. **Skew Bridges**

For skew bridges, detailed analysis of forces shall be required. However, if the angle of skew is within 15^0, such detailed analysis may not be necessary.

508. DESIGN OF BEAMS

508.1. **General**

508.1.1. Beams are defined as members with solid webs or with openings, subjected primarily to bending, including members of rolled and hollow section, plate girders and box girders.

508.1.2. Beams shall be proportioned on the basis of the moment of inertia of the gross cross-section with the neutral axis taken at the centroid of that section. In computing the maximum stresses, the stresses calculated on this basis shall be increased in the ratio of the gross to the effective area of the flange section. For this purpose, the flange sectional area in riveted or bolted construction shall be taken to be that of the flange plates, flange angles and the portion of the web and side plates, if any, between the flange angles. In welded construction, the flange sectional area shall be taken to be that of flange plates and of the tongue plates (i.e., the thick vertical plates connecting flange to web), if any, upto a depth of the tongue plate equal to eight times its thickness, which shall not be less than twice that of the web.

508.2. **Web Plates**

508.2.1. **Minimum thickness:** The thickness of the web plate shall conform to the requirements of Clause 507.7 and further shall not be less than the following :

(a) *for unstiffened webs :*

$d_1/85$ for steel conforming to IS:2062

$d_1/75$ for steel conforming to IS:8500

(b) *for vertically stiffened webs :*

1/180 of the smaller clear panel dimension.

l/270 of the greater clear panel dimension

$d_2/200$ for steel conforming to IS:2062 or

$d_2/180$ for steel conforming to IS:8500

(c) *for webs stiffened, both vertically and horizontally* and with the horizontal stiffener at distance from the compression flange of 2/5 of the distance from the compression flange to the neutral axis :

l/180 of the smaller clear panel dimension,

l/270 of the greater clear panel dimension, and

$d_2/250$ for steel conforming to IS:2062 or

$d_2/225$ for steel conforming to IS:8500

(d) when there is also a horizontal stiffener at the neutral axis of the girder

l/180 of the smaller clear panel dimension,

l/270 of the greater clear panel dimension, and

$d_2/400$ for steel conforming to IS:2062 or

$d_2/360$ for steel conforming to IS:8500

In the above, d_2 is the clear distance between flange angles or, where there are no flange angles, between flanges (ignoring fillets); but where tongue plates having a thickness not less than twice the thickness of the web plate are used, d_1 is the depth of the girder between the flanges less the sum of the depths of the tongue plates or eight times the sum of the thicknesses of the tongue plates, whichever is less, and d_2 is twice the clear distance from the compression flange angles or plate, or tongue plate to the neutral axis.

508.3. **Flanges**

508.3.1. The effective sectional area of compression flanges shall be the gross area with the specified deduction for excessive width of plates (Clause 508.3.3., 508.3.4) and maximum deduction for open holes and holes for bolts occurring in section perpendicular to the axis of the member.

508.3.2. The effective sectional area of tension flanges shall be the gross sectional area with specified deduction for excessive width or projection of plates (Clause 508.3.5) and deduction of all holes as specified for rivet or bolt holes in tension members in Clause 507.8.2.1.

508.3.3. In riveted or bolted construction, flange angles shall form as large a part of the area of the flange as practicable (preferably not, less than 1/3) and the number of flange plates shall be kept to a minimum. Where flange plates are used, they shall preferably of equal thickness and atleast one plate of the top flange shall extend the full length of the girder, unless the top edge of the web is finished flush with the flange angles.

Compression flange plates unstiffened at their edges shall not project beyond the outer lines of connections to the flange angles by more than 16t' for steel conforming to IS:2062 or 14t' for steel conforming to IS:8500 where t' is the thickness of the thinnest flange plate or the aggregate thickness of the two or more plates when projecting portions of these plates are adequately tacked together.

508.3.4. In welded construction, compression flange plates unstiffened at their edges shall not project beyond the line of connection to the web or tongue plates by more than 12t'.

508.3.5. In all cases tension flange plates, stiffened or unstiffened at their edges, shall not project beyond the outer line of connections to the flange angles (or where there are no flange angles, to the web or tongue plates) by more than 20t'.

508.3.6. For the flanges of beams with vertical stiffeners only (see Clause 508.11.2.2), where d1/t is greater than 130 in the case of steel conforming to IS:2062 or 110 in the case of steel conforming to IS:8500 and when the average shear stress in the web is greater than 0.6 of the permissible stress given for mild steel in Clause 506.4.1, the quantity $I/(b^3t)$ shall not be less than 2.5×10^{-4} in the case of steel conforming to IS:2062 and 3×10^{-4} in the case of steel conforming to IS:8500.

where,

I= the moment of inertia of the compression flange about its axis normal to the web, taken as that of the flange angles and plates, and the enclosed portion of the web in the case of riveted construction, and in the case of welded construction, as the flange plate together with a depth of web (adjacent to the flange plate) equal to 16 times the web thickness.

d_1 =effective depth of the girder as defined in Clause 508.2.1

b = spacing of stiffeners

t = thickness of web

508.4. Effective Length of Compression Flanges

The effective length of the compression flange for buckling normal to the plane of the girder shall be as given below.

508.4.1. Simply supported beams with no intermediate lateral support to compression flange, but with each end restrained against torsion.

508.4.1.1. When there is no intermediate lateral restraint to a compression flange, effective length l should be taken as

$$l = k l\ L$$

where,

L = span of the beam (i.e. between restraint at supports)

$k l$ = 1.0 if the compression flange is free to rotate in plan at the points of support, or

= 0.85 if the compression flange is partially restrained against rotation in plan at one support and free to rotate in plan at the other, or

= 0.7 if the compression flange is fully restrained against rotation in plan at the points of support.

508.4.1.2. Restraint against torsion at the supports can be provided by web or flange cleats, by bearing stiffeners, by end frames or by lateral supports to the compression flange. The restraint element shall be designed to resist, in addition to the effects of wind and other applied lateral forces, the effects of a horizontal force acting normal to the compression flange of the girder at the level of the centroid of this flange where

$$F = \frac{1.4 \times 10^{-3} \times l}{\delta(f_{cb}/f_{bc} - 1.7)}$$

In the above formula :

l = has the value given in Clause 508.4.1.1.

f_{cb} = the critical stress in the flange as defined in Clause 508.6.2

f_{bc} = the calculated working stress in flange

δ = the deflection of the flange under the action of unit horizontal force as defined in Clause 508.4.2

508.4.2. Simply supported beams with compression flange laterally supported by *U-Frames*.

For simply supported girders where there is no lateral bracing of the compression flanges but where cross members and stiffeners forming U-Frames provide lateral restraint.

$$l = 2.5 \times \sqrt[4]{(E\,I_c\,a\;\delta)}$$ but not less than "a"

where,

E = Young's Modulus

I_c = maximum moment of inertia of compression flange about its centroidal axis parallel to the web of the girder

a = distance between frames

δ = the lateral deflection which would occur in the U-Frame at the level of the centroid of the flange being considered when a unit force acts laterally to the U-Frame only at this point and simultaneously at each corresponding point on the other flange or flanges connecting to the same U-Frame. The direction of each unit force should be such as to produce the maximum aggregate value of δ. The U-Frame should be taken as fixed in position at each point or intersection between the cross member and a vertical as free and unconnected at all other points.

when δ is not greater than $a^3/(40\,E\,I_c)$

$l = a$

In cases of symmetrical U-Frames where cross members and stiffners are each of constant moment of inertia throughout their own length.

$$\delta = \frac{(d_1)^3}{3EI_1} + \frac{(d_2)^2 b}{EI_2}$$

where,

d_1 = distance of the centroid of the compression flange from the top of the cross member

d_2 = distance of the centroid of the compression flange from the neutral axis of the cross member

b = half the distance between centres of the main girders

I_1 = the moment of inertia of a pair of stiffeners about the centre of the web, or a single stiffener about the face of the web. A width of web plate upto 16 times the web thickness may be included on each side of centerline of connection

I_2 = moment of inertia of the cross member in its plane of bending

508.4.3. **Beams with laterally supported compression flanges:** When a compression flange is provided with effective discrete lateral restraints effective length l should be taken as the greatest distance centre to centre of restraint members between a restraint and a support. Where such restraint is provided by interconnecting bracing, consideration should be given to the possibility of lateral instability of the combined cross-section.

508.4.4. **Cantilever beams without intermediate lateral support:** When a cantilever beam is not provided with lateral support between its support and tip, l may be taken from Table 8.1 where L is the length of cantilever.

Table 8.1. Effective Length l for a Cantilever Beam without Intermediate Lateral Restraint

(Clause 508.4.4)

Restraint Conditions		Position of load	
At support	At tip	On tension flange where there is no lateral restraint to load or flange	Al other position
1. Built in	(a) Free (b) Tension flange held against displacement (c) Both flanges held against lateral displacement	1.4 L 1.4 L 0.6 L	0.8 L 0.7 L 0.6 L
2. Continuous and both flanges held against lateral displacement	(a) Free (b) Tension flange held against displacement (c) Both flanges held against lateral displacement	2.5 L 2.5 L 1.5 L	1.0 L 0.9 L 0.8 L
3. Continuous with tension flanges held against lateral displacement	(a) Free (b) Tension flange held against displacement (c) Both flanges held against lateral displacement	7.5 L 7.5 L 4.5 L	3.0 L 2.7 L 2.4 L

Note : L is the length of the cantilever

508.4.5. **Beams continuously restrained by deck at compression flange level:** A compression flange continuously supporting a reinforced concrete or steel deck shall be deemed to be effectively restrained laterally throughout its length (i.e., l = 0) if the frictional or positive connection of the deck to the flange is capable of resisting a lateral force of 2.5 per cent of the force in the flange at the point of maximum bending moment, distributed uniformly along length.

508.5. **Slenderness Ratio**

The slenderness ratio λ *(i.e.,* l / r_{yy}*)* of a beam shall not exceed 300 and it shall not exceed 150 for cantilevers.

Where, l = effective length of the compression flange as specified in Clause 508.4.

r_{yy} = the radius of gyration of the whole beam about its y-y axis based on the gross moment of inertia and the gross sectional area.

508.6. **Permissible Bending Stresses**

508.6.1. The tensile and compressive bending stresses calculated according to Clause 508.1.2 shall not exceed the appropriate permissible stresses in Table 6.2.

508.6.2. For beams and plate girders with I smaller than I_x

where,

I_y = moment of inertia of the whole section about the axis lying in the plane of bending (y-y axis)

I_x = moment of inertia of the whole section about the axis normal to the plane of bending (x-x axis)

The bending compression stress calculated according to Clause 508.1.2 shall not exceed the permissible bending compressive stress σ_{bc} given in Table 8.2 corresponding to f_{cb}, (elastic critical stress), calculated as follows :

Table 8.2. Values of σ_{bc} Calculated from f_{cb} for Different Values of f_y

(Clause 508.6.2)

All Units in MPa

$f_y \rightarrow$ $f_{cb} \downarrow$	250	340	400
20	13	13	13
30	19	19	19
40	25	26	26
50	31	31	32
60	36	37	38
70	41	43	44
80	46	48	49
90	51	54	55
100	55	59	60
110	60	64	65
120	64	68	70
130	67	73	75
140	71	77	80
150	74	81	84
160	78	85	89
170	81	89	93
180	84	93	97
190	87	97	102
200	89	100	105
210	92	103	109
220	94	106	112
230	96	110	116

240	99	113	119
250	101	115	122
260	103	118	126
270	104	121	129
280	106	123	132
290	108	126	135
300	110	128	137
310	111	130	140
320	113	133	143
330	114	135	145
340	115	137	148
350	117	139	150
360	118	141	152
370	119	143	155
380	120	144	157
390	121	146	159
400	122	148	161
420	124	151	165
440	126	154	169
460	128	157	172
480	130	159	175
500	131	162	178
520	133	164	181
540	134	166	184
560	135	168	187
580	136	170	189
600	137	172	192
620	138	174	194
640	139	175	196
660	140	177	198
680	141	178	200

700	142	180	202
720	143	181	204
740	143	182	205
760	144	184	207
780	145	185	208
800	145	186	210
850	147	188	213
900	148	191	216
950	149	193	219
1000	150	195	222
1050	151	196	224
1100	152	198	226
1150	152	199	228
1200	153	200	230
1300	154	203	233
1400	155	205	236
1500	156	206	238
1600	157	208	240
1700	157	209	242
1800	158	210	243
1900	158	211	245
2000	159	212	246
2200	160	213	248
2400	160	215	250
2600	161	216	251
2800	161	216	252
3000	161	217	253
3500	162	218	255
4000	163	219	257
4500	163	220	258
5000	163	221	259
5500	163	221	259
6000	164	222	260

Elastic critical stress

The elastic critical stress f_{cb} for beams and plate girders with Iy smaller than Ix shall be calculated using the following formula :

$$f_{cb} = k_1 (X + k_2 Y)(c_2 / c_1)$$

where,

$$X = Y \sqrt{1 + (1/20)[(l T)/(r_y D)]^2} \quad MPa$$

$$Y = 26.5 \times 10^5 / (l / r_y)^2 \ MPa$$

c_1, c_2 = respectively the lesser and greater distances from the section neutral axis to the extreme fibres

D = overall depth of the beam

T = mean thickness of the compression flange

l = effective length of compression flange

r_y = radius of gyration of the section about its axis of minimum strength (y-y axis)

k_1 = a co-efficient to allow for reduction in thickness or breadth of flanges between the points of effective lateral restraint and depends on ψ the ratio of total area of both flanges at the point of least bending moment to the corresponding area at the point of greatest bending moment between such points of restraint. Values of k_1 for different values of ψ are given in Table 8.3.

k_2 = a co-efficient to allow for the inequality of flanges and depends on ϖ, the ratio of the moment of inertia of the compression flange alone to that of the sum of the moments of inertia of the flanges each calculated about its own axis parallel to the axis of the girder, at the point of maximum bending moment. The values of k_2, for different values ϖ of are given in Table 8.4. Values of X and Y for appropriate values of D/T and l/r_y are given in Table 8.5.

Table 8.3. Value of k_1 for Beams with Curtailed Flanges

(Clause 508.6.2)

ψ	1.0	0.9	0.8	0.7	0.6	0.5	0.4	0.3	0.2	0.1	0.0
k_1	1.0	1.0	1.0	0.9	0.8	0.7	0.6	0.5	0.4	0.3	0.2

Note : Flanges should not be reduced in breadth to give a value of lower than 0.25

Table 8.4. Values of k_2 for Beams with Unequal Flanges

(Clause 508.6.2)

β	1.0	0.9	0.8	0.7	0.6	0.5	0.4	0.3	0.2	0.1	0.0
k_2	0.5	0.4	0.3	0.2	0.1	0	-0.2	-0.4	—.6	-0.8	-1.0

508.6.2.1. Values of f_{cb} shall be increased by 20 per cent when T/t is not greater than 2.0 and d_1/t is not greater than $1344/\sqrt{f_y}$ where d_1 is as defined in Clause 508.2.1 and t the thickness of the web and the value of f_y is expressed in MPa.

508.6.3. **Beams bent about the axis of minimum strength (y-y axis):** The maximum permissible bending stress in tension or in compression in beams bent about the axis of minimum strength shall not exceed the appropriate permissible stresses in Table 6.2.

508.6.4. **Angle and tee shapes:** The bending stress in the leg when loaded with the flange or table in compression shall not exceed the appropriate permissible stresses in Table 6.2. When loaded with the leg in compression, the permissible bending stress shall be calculated from Clause 508.6.2 with k_2 = -*1.0* and T = thickness of leg.

Table 8.5. Values of *X* and *Y* for Calculating f_{cb}

(Clause 508.6.2)

	X															Y
D/T → f_y ↓	8	10	12	14	16	18	20	25	30	35	40	50	60	80	100	
40	2484	2222	2066	1965	1897	1849	1814	1759	1728	1709	1697	1683	1675	1667	1663	1656
45	2103	1856	1709	1612	1546	1409	1465	1411	1380	1362	1349	1335	1327	1319	1315	1309
50	1822	1590	1449	1357	1293	1248	1214	1161	1131	1113	1101	1086	1078	1070	1067	1060
55	1607	1389	1234	1166	1105	1060	1028	976	947	929	917	902	894	886	883	876
60	1437	1232	1104	1020	961	918	886	835	806	788	776	762	754	746	743	736
65	1301	1107	985	904	847	806	775	726	697	679	657	653	645	637	634	627
70	1188	1005	889	811	757	717	687	638	610	592	581	567	559	551	547	541
75	1094	920	810	735	682	644	615	567	540	522	511	467	489	481	478	471
80	1014	849	743	672	621	584	556	509	482	465	454	440	432	424	421	414
85	945	788	687	618	570	533	506	461	434	417	406	392	385	377	373	367
90	886	735	639	573	526	491	464	420	394	377	366	353	345	337	373	327
95	833	689	597	534	488	454	428	385	360	343	332	319	311	304	300	294
100	787	649	560	499	455	423	398	356	331	314	304	290	283	275	272	265
110	708	582	499	443	402	371	347	307	283	268	257	244	237	229	226	219
120	644	527	451	398	359	330	308	270	247	232	222	209	202	194	191	184
130	591	482	411	361	325	298	277	240	218	204	194	181	174	167	163	157
140	546	444	378	331	297	271	251	217	195	181	172	160	153	145	142	135
150	508	412	350	306	274	249	230	197	177	163	154	142	135	145	124	118
160	474	385	326	284	254	230	212	181	161	148	139	127	121	113	110	104
170	445	360	306	265	236	214	197	167	148	135	126	115	109	102	95	92
180	420	339	286	249	221	200	184	155	137	125	116	105	98	92	88	82
190	397	320	270	235	208	188	172	145	127	115	107	96	90	83	80	73
200	476	304	256	222	197	177	162	136	119	107	99	89	83	76	78	66
210	358	238	243	210	186	168	153	128	112	101	93	82	76	70	66	60
220	341	275	231	200	177	159	145	121	105	94	87	77	71	64	61	55
230	326	262	220	191	169	152	138	115	99	89	82	72	66	60	56	50
240	312	251	211	182	161	145	132	109	94	84	77	67	62	55	52	46
250	299	241	202	175	151	139	126	104	90	80	73	64	58	52	49	42
260	288	231	194	167	148	133	121	99	85	76	68	60	55	48	45	39
270	277	222	186	161	142	127	115	95	82	72	66	57	52	46	42	36
280	267	214	180	155	137	122	111	91	78	69	63	54	49	43	40	34
290	257	207	173	149	132	118	107	88	75	66	60	52	46	41	38	32
300	249	200	167	144	127	114	103	84	72	64	57	49	44	38	35	30

Note— Intermediate values may be obtained by linear interpolation.

508.7. **Permissible Shear Stress**

508.7.1. **Maximum shear stress:** The maximum shear stress in a member having regard to the distribution of stresses in conformity with the elastic behaviour of the member in flexure, shall not exceed the appropriate permissible stress in Table 6.2.

508.7.2. **Average shear stress:** The average shear stress in a member calculated on the cross-section of the web shall not exceed the maximum permissible average shear stress $_{va}$ as follows :-

(a) *For unstiffened webs*: Appropriate permissible stress in Table 6.2.

(b) *For stiffended webs:* The values given in Tables 8.6, 8.7 and 8.8 as appropriate yield stress values 250, 340 and 400 MPa respectively.

Note : The allowable stresses given in Tables 8.6, 8.7 and 8.8 apply provided any reduction of the web cross-section is due only to rivet/bolt holes, etc. where large apertures are cut in the web, a special analysis shall be made to ensure that the maximum permissible average shear stresses laid down in this standard are not exceeded.

Table 8.6. Permissible Average Shear Stress τ_{va} in Stiffened Webs of Steel with f_y=250 Mpa

(Clause 508.7.2)

Strees τ_{va} (Mpa) for different distances between stiffeners

d/t	0.3d	0.4d	0.5d	0.6d	0.7d	0.8d	0.9d	1.0d	1.1d	1.2d	1.3d	1.4d	1.5d
90	100	100	100	100	100	100	100	100	100	100	100	100	100
95	100	100	100	100	100	100	100	100	100	100	100	100	99
100	100	100	100	100	100	100	100	100	100	100	99	99	98
105	100	100	100	100	100	100	100	100	100	99	98	97	96
110	100	100	100	100	100	100	100	100	99	98	96	95	94
115	100	100	100	100	100	100	100	100	98	96	95	94	93
120	100	100	100	100	100	100	100	98	96	95	93	92	91
125	100	100	100	100	100	100	98	97	95	93	92	91	90
130	100	100	100	100	100	99	97	96	94	92	90	89	88
135	100	100	100	100	100	98	96	94	92	90	89	87	86
140	100	100	100	100	99	96	95	93	91	89	87	86	85
150	100	100	100	100	97	94	92	90	88	85	84	83	81
160	100	100	100	98	94	92	89	88	85	83	81	80	78
170	100	100	100	96	92	89	87	85	82	80	78	76	75
180	100	100	98	94	90	87	84	82	80	77	75	73	72
190	100	100	97	92	88	84	82						
200	100	100	95	90	86	82	81						
210	100	99	93	88	83	81							
220	100	98	91	86	81	80							
230	100	96	90	84	79								
240	100	95	88	83	77								
250	100	93	86	82	74								
260	100	92	85	81									
270	100	90	84	81									

Non-applicable zone

Note— Intermediate values may be obtained by linear interpolation.

Table 8.7. Permissible Average Shear Stress τ_{va} in Stiffened Webs of Steel with f_y=340 Mpa

(Clause 508.7.2)

Strees τ_{va} (Mpa) for different distances between stiffeners

d/tw	0.3d	0.4d	0.5d	0.6d	0.7d	0.8d	0.9d	1.0d	1.1d	1.2d	1.3d	1.4d	1.5d
75	136	136	136	136	136	136	136	136	136	136	136	136	136
80	136	136	136	136	136	136	136	136	136	136	136	136	136
85	136	136	136	136	136	136	136	136	136	136	136	134	133
90	136	136	136	136	136	136	136	136	136	135	133	132	131
95	136	136	136	136	136	136	136	136	135	133	131	129	128
100	136	136	136	136	136	136	136	136	132	130	128	127	126
105	136	136	136	136	136	136	135	133	130	128	126	124	123
110	136	136	136	136	136	135	133	131	128	126	124	122	120
115	136	136	136	136	136	133	131	129	126	123	121	119	118
120	136	136	136	136	135	131	129	127	124	121	119	117	115
125	136	136	136	136	133	129	127	125	121	119	116	114	113
130	136	136	136	135	131	127	125	122	119	116	114	112	110
135	136	136	136	134	129	126	123	120	117	114	111	109	108
140	136	136	136	132	127	124	121	118	115	112	109	107	105
150	136	136	136	129	124	120	117	114	110	107	104	102	100
									Non-applicable zone.				
160	136	136	132	126	120	116	113	110				97	95
170	136	136	129	123	117	112	109	106	101	98	95	92	90
180	136	135	127	119	113	108	105	102	97	93	90	87	84
190	136	133	124	116	110	105	100						
200	136	130	121	113	106	101	96						
210	136	128	118	110	103	97							
220	136	126	116	107	99	93							
230	136	123	113	103	96								
240	134	121	110	100	92								
250	132	119	107	97	89								
260	130	116	104	94									
270	128	114	102	91									

Note— Intermediate values may be obtained by linear interpolation.

Table 8.8. Permissible Average Shear Stress τ_{va} in Stiffened Webs of Steel with f_y=400 Mpa

(Clause 508.7.2)

Strees τ_{va} (Mpa) for different distances between stiffeners

d/t↓	0.3d	0.4d	0.5d	0.6d	0.7d	0.8d	0.9d	1.0d	1.1d	1.2d	1.3d	1.4d	1.5d
70	160	160	160	160	160	160	160	160	160	160	160	160	160
75	160	160	160	160	160	160	160	160	160	160	160	160	159
80	160	160	160	160	160	160	160	160	160	160	159	157	156
85	160	160	160	160	160	160	160	160	160	158	156	154	152
90	160	160	160	160	160	160	160	160	157	155	152	151	149
95	160	160	160	160	160	160	159	157	154	152	149	147	146
100	160	160	160	160	160	160	157	155	151	149	146	144	143
105	160	160	160	160	160	157	154	152	149	146	143	141	139
110	160	160	160	160	159	155	152	149	146	143	140	138	136
115	160	160	160	160	156	152	149	147	143	140	137	135	133
120	160	160	160	159	154	150	147	144	140	137	134	132	129
125	160	160	160	157	152	147	144	141	137	134	131	128	126
130	160	160	160	155	150	145	141	139	131	131	128	125	123
135	160	160	160	153	147	143	139	136	132	128	125	122	120
140	160	160	158	151	145	140	136	133	129	125	122	119	116
150	160	160	155	147	141	135	131	128	123	119	115	112	110
										Non-applicable zone			
160	160	160	151	143	136	130	126	123	117	---	---	---	103
170	160	153	148	139	132	126	121	117	112	107	103	100	97
180	160	155	144	135	127	121	116	112	106	101	97	93	90
190	160	152	140	131	123	116	111						
200	160	149	137	127	118	111	106						
210	160	146	133	123	114	106							
220	157	143	130	119	109	101							
230	155	140	126	114	105	143							
240	153	137	123	110	100	140							
250	151	134	119	106	96	135							
260	148	131	116	102		130							
270	146	128	112	98									

Note— Intermediate values may be obtained by linear interpolation.

508.8. **Curtailment of Flange Plates**

Each flange plate shall be extended beyond its theoretical cut-off point adequately. The extension shall contain sufficient rivets, bolts and welds to withstand the forces developed at the theoretical cut-off point.

In welded construction, the use of curtailed flange plates shall be avoided as far as possible, local strengthening being provided by other means, such as, inserting by butt welding a thicker and or wider plate. The heavier section plate shall be suitably tapered to the lighter plate. If, in welded construction the use of curtailed flange plates cannot be avoided, the end of the plate shall be tapered in plan to a rounded end and all welds shall be continuous round the ends.

508.9. **Connection of Flanges to Web**

The flanges of plate girders shall be connected to the web by sufficient rivets. bolts or welds to transmit the horizontal shear force combined with any vertical loads which are directly applied to the flange. In welded construction, where the web is in close contact with the flange before welding, vertical loads causing compression may be deemed to be resisted by the bearing between the flange and the web.

508.10. **Dispersion of Load Through Flange to Web**

Where a load is directly applied to a flange, it shall be considered as dispersed uniformly through the flange and the web at an angle of 45°.

508.11. **Web Stiffeners**

Web stiffeners shall be provided as follows :

508.11.1. **Load bearing stiffeners**

508.11.1.1 **General:** Webs of plate girders and rolled beams shall be provided with load bearing stiffeners at points of supports and at points of concentrated load where reaction or concentrated load exceeds the value of

$$\sigma_{ac} . t . B$$

where,

σ_{ac} = maximum permissible axial stress for struts as given in Clause 506.4.2.1 for a slenderness ratio of $(d_{3} \sqrt{3})/t$

t = web thickness

d_{3} = clear depth of web between root fillets

B = the length of the stiff portion of the bearing plus the additional length given by dispersion at 45° to the level of the neutral axis. The stiff portion of a bearing is that length which cannot deform appreciably in bending and shall not be taken as greater than half the depth of the beams continuous over a bearing.

508.11.1.2. **Details and design**

(a) Load bearing stiffeners should be in pairs (i.e., two legs of plates, angles, etc.) placed symmetrically at both sides of the web. When the condition is not met the effect of the resulting eccentricity should be considered.

(b) The ends of the load bearing stiffener should be closely fitted or adequately connected to both flanges. They should be shaped to allow space for any root fillet or weld connecting the web to the flange, with a clearance not exceeding five times the thickness of the web.

(c) Load bearing stiffeners shall not be joggled and shall be solidly packed throughout.

(d) Outstanding legs or each pair of load bearing stiffeners shall be so proportioned that the bearing stress on that part of their area in contact with the flange and clear of the root of the flange or flange angles or clear of the flange welds, does not exceed the bearing stress specified in Clause 506.4.1.

(e) Load bearing stiffeners consisting of two legs shall be designed as struts assuming the section to consist of the pair of stiffeners together with a length of web on each side of the centre line of the stiffeners equal to twenty times the web thickness (but limited to the edge distance of the web and half the distance of the adjacent stiffener).

In case of bearing stiffeners consisting of four or more legs, the effective stiffener section should be taken to comprise the stiffeners, the web plate between the two outer legs and a portion of web plate not exceeding the length of the web as specified for single leg stiffeners on the outer sides of the outer legs.

(f) The radius of gyration shall be taken about the axis parallel to the web of the beam or girder, and the working stress shall be in accordance with appropriate allowable value for a strut, assuming the effective length equal to 0.7 times the length of the stiffener.

(g) The load bearing stiffeners shall be provided with sufficient rivets, bolts or welds to transmit to the web the whole of the load in the stiffeners.

(h) When load bearing stiffeners at supports are the sole means of providing restraint against torsion (see Clause 508.4.1.2) the moment of inertia I of the stiffener shall not be less than

$(D^3 T_m / 250) \times (R / W)$

where, I = moment of inertia of the pair of

stiffeners about the centre line of the web plate.

D = overall depth of the girder

T_m = maximum thickness of compression flange

R = reaction of the bearing

W = total load on girder

(i) In addition, the base of the stiffeners in conjunction with the bearing of the girder shall be capable of resisting a moment due to horizontal force specified in Clause 508.4.1.3.

508.11.2. **Intermediate stiffeners**

508.11.2.1. **General:** When the thickness of the web is less than the limits specified in Clause 508.2.1. (a), transverse stiffeners shall be provided throughout the length of the girder. When the thickness of the web is less than the limits specified in Clause 508.2.1. (b), longitudinal stiffeners shall be provided in addition to the transverse stiffeners.

In no case shall the greater unsupported clear dimension of a web panel exceed 270 t nor the lesser unsupported clear dimension of the same panel exceed 180 t where t is the thickness of the web plate.

508.11.2.2. **Transverse stiffeners:** Where transverse stiffeners are required, they shall be provided throughout the length of the girder at a distance apart not greater than 1.5 d_1 and not less than 0.33 d_1, where d_1 is the depth as defined in Clause 508.2.1. Where horizontal stiffeners are provided d_1 shall be taken as the clear distance between the horizontal stiffener and the farthest flange ignoring fillets.

Transverse-stiffeners shall be designed so that I is not less than :

$$1.5 \times (d_1^3 \times t^3) / S^2$$

where,

I	=	the moment of inertia of a pair of stiffeners about the centre of the web or a single stiffener about the face of the web
t	=	minimum required thickness of the web
S	=	the maximum permitted clear distance between stiffener for thickness t

Note : If the thickness of the web is made greater, or the spacing of stiffener made smaller than that required by the standard, the moment of inertia of the stiffener need not be correspondingly increased.

Intermediate transverse stiffeners, when not acting as load bearing stiffeners, may be joggled and may be single or in pairs placed one on each side of the web. Where single stiffeners are used, they should preferably be placed alternatively on opposite sides of the web. The stiffeners shall extend from flange to flange. They can be connected or fitted to, or kept well clear of the flanges.

508.11.2.3. **Longitudinal stiffeners**

Where longitudinal stiffeners are used in addition to vertical stiffeners they shall be as follows:

One longitudinal stiffener, on one or both sides of the web, shall be placed at a distance from the compression flange equal to two fifths of the distance from the compression flange to the neutral axis, when the thickness of the web is less than $d_2/200$ for steel conforming to IS:2062 and $d_2/180$ for steel conforming to IS:8500 where d_2 is the depth of web as defined in Clause 508.2.1. This stiffener shall have a moment of inertia I not less than $4\ S_1\ t^3$ where I and t are as defined in Clause 508.11.2.2 and S_1 is the actual distance between stiffeners.

A second longitudinal, on one or both sides of the web shall be

placed on the neutral axis of the girder when the thickness of the web is less than $d_2/250$ for steel conforming to IS:2062 or $d_2/225$ for steel conforming to IS:8500. The stiffener shall have a moment of inertia I not less than $d_2 t^3$ where I and t are as defined in Clause 508.11.2.2 and d_2 in Clause 508.2.1.

Longitudinal stiffeners shall extend between vertical stiffeners but need not be continuous over them or connected to them.

508.11.2.4. **External forces on intermediate stiffeners:** When vertical intermediate stiffeners are subject to bending moments and shears due to the eccentricity of vertical loads, or the action of transverse forces, the moment of inertia I of the stiffeners specified in Clause 508.11.2.2 shall be increased as follows:

(a) Bending moment on stiffener due to eccentricity of vertical loading with respect to the vertical axis of the web :

Increase of I $= (1.5\ M\ D^2)\ /\ (E\ t)$

(b) Lateral loading on stiffener

Increase of I $= (3\ P\ D^3)\ /\ (E\ t)$

where,

M = the applied bending moment

D = overall depth of girder

E = Young's modulus

t = thickness of web

P = lateral force to be taken by the stiffener and

deemed to be applied at the compression flange of the girder.

508.11.2.5. **Connection of intermediate stiffeners to web:** Intermediate transverse and longitudinal stiffeners not subjected to external loads shall be connected to the web by welds or rivets, in order to withstand a shearing force in kilograms per millimetre run between each component of stiffener and the web, of not less than $12.6.t^2/h$, where, t equals web thickness in mm and h equals the projection in mm, of the stiffener component from the web.

508.11.2.6. **Outstand of all stiffeners:** Unless the outer edge of each stiffener is continuously stiffened, the outstand from the web shall not be more than the following :

For sections :	16t for steel conforming to IS:2062
	14t for steel conforming to IS:8500
For flats :	12t for all steels

Where t is the thickness of the section or flat.

508.12. **Flange Splices**

Flange joints should preferably not be located at points of maximum stress. Where splice plates are used, their area shall be atleast 5 per cent in excess of the area of the flange element spliced, their centre of gravity shall coincide, as nearly as possible, with that of the element spliced. There shall be enough rivets or welds on each side of the splice to develop the load in the element spliced plus 5 per cent, but in no case should the strength developed be less than 50 per cent of the effective strength of the material spliced.

In welded construction, flange plates or angles shall be joined by full penetration butt welds wherever possible. These butt welds shall develop the full strengths of plates or angles. Where this is not possible, splice plate should be used.

508.13. **Splices in Webs**

Splices in the webs of plate girders and rolled sections shall be designed to resist the shears and moments at the spliced section.

In riveted construction, splice plates shall be provided on each side of the web. In welded construction, web plates shall be joined by full penetration butt welds wherever possible. Where this is not possible, splice plate may be used on both sides.

508.14. **End Connections**

Connections at the ends of all beams designed as simply supported beams shall have flexibility to take angular deffection..

508.15. **Lateral Bracing**

All spans shall be provided with a lateral bracing system extending from end to end of sufficient strength to transmit to the bearings all lateral forces due to wind, seismic effect etc., as applicable.

508.16. **Expansion and Contraction**

In all bridges, provision shall be made in the design to resist thermal stresses induced, or means shall be provided for movement caused by temperature changes. Provision shall also be made for changes in length of span resulting from live loads.

509. DESIGN OF COMPRESSION MEMBERS

509.1. General

Design of compression members should generally follow the considerations under Clause 511.3 under "Trusses or Open Web Girders" of this Code.

509.2. Base Plate

509.2.1. Base plate is a structural part which serves as medium for uniformly transferring load from member/stanchion/column to foundation.

509.2.2. Area of base plate should be such that at any point reactive pressure acting on it is less than allowable stress of concrete in compression.

$$A_F = N / \sigma_{cc}$$

where,

A_F = Area of base plate

N = Load in the member

σ_{cc} = Allowable compressive stress of concrete

For (Crushing value of concrete) IS:456 may be referred for guidance.

509.2.2.1. Width of base plate should be $B = b(\text{ or } h) + 2t_s + 2c$,

where,

b and h = Size of member/stanchion/column,

t_s = Thickness of saddle plate, 8 - 10 mm

c = Cantilever portion restricted to 100 - 120 mm, but not less than 20 mm from outside member, stiffener to the edge of base plate.

Length of base plate $L = \dfrac{A_F}{B}$

A_F = Area of base plate

509.2.2.2. Thickness of base plate should not be less than 20 mm.

509.2.2.3. Thickness of plate is determined from its bending consideration due to reactive pressure of foundation on base plate.

$$P_F = N / A_F$$

where,

P_F = Reactive pressure on base plate

Base plate area in general can be divided in four types depending upon boundary conditions of support (stiffeners).

(i) Cantilever

(ii) Supported on two sides perpendicular to each other

(iii) Supported on three sides

(iv) Supported on four sides

509.2.2.3.1. Bending moment in case of cantilever for 1 cm width of base plate (case-1) :

$$M_1 = P_F c^2 / 2$$

509.2.2.3.2. Maximum bending moment in centre of free B-side in cases of plate having support at three sides and also at two perpendicular sides:

$$M_2 = P_F b^2$$

where,

b = Length of free shorter side of plate

α = Coefficient as per table below depending on a/b.

a/b	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	2.0	More than 2
α	0.06	0.074	0.088	0.097	0.107	0.112	0.120	0.126	0.132	0.133

If $a/b \leq 0.5$ the support of plate on a-side does not have any effect, as such for bending moment on base plate formula for cantilever type should be used with $c = a$.

509.2.2.3.3. Maximum Bending Moment in case of plate having support at four sides.

$$M_{t} = \beta P_{F} (b_{l})^{2}$$

where,

b_l = is short side length

β = Coefficient as per table below, depending on a_1/b_1.

a_l/b_l	1.0	1.1	1.2	1.3	1.4	1.5	1.6	1.7	1.8	1.9	2.0	More than 2
β	0.048	0.055	0.063	0.069	0.075	0.081	0.086	0.091	0.094	0.098	0.1	0.125

In case of $a_l/b_l > 2$, plate works as single span simply supported beam and bending moment,

$$M_{3} = P_{F} (b_{l})^{2} / 8$$

509.2.2.3.4. Thickness of the base plate $t_{B} = \sqrt{\frac{6M}{\sigma}}$

where,

M = Maximum bending moment considering all areas in which base plate is divided.

σ_{bc} = Maximum permissible bending stress in slab base.

509.2.2.4. Section of stiffeners saddle element of base plate and its connections are designed for loads coming on them. Main stiffeners are designed as simply supported over hanging beam loaded with uniformly distributed load equal to $qs_{l} = P_{F} x\, l_{l}$ and checked for bending and shear stresses.

509.2.2.4.1. Secondary stiffeners (considering cantilever) are designed for load equal to $qs_2 = P_F \times l_2$ and checked for bending and shear stress.

509.2.2.5. Welded and riveted connections are designed to transfer the loads coming on stiffeners to main member and also to connect base plate with stiffeners.

509.2.2.6. In case of heavy load transfer from member to the base plate machining of contact surface between base plate and member is recommended and the area of the base plate shall be sufficient to limit the stress in bearing for whole of the load. In such cases, the weld or rivet connecting base plate with stiffeners and main member should be designed for 25 per cent of total load coming on base plate (for resisting the unforeseen bending and shear) which should be resisted by total weld length or all rivets.

509.2.2.7. Base plate for eccentrically loaded members - Action due to bending moment in base plate along with compression causes non-uniform pressure on the foundation and value of max. and min. pressure can be computed as under:

$$P_{F Max,\, Min} = \frac{N}{BL} \pm \frac{6M_x}{BL^2} \pm \frac{6My}{LB^2}$$

where, B & L are width and length of base plate. M_x and M_y are moments in the length and width direction of base plate respectively.

509.2.2.8. Thickness of base plate is computed as per Clauses 509.2.2.3.1 to 509.2.2.3.4 and bending moment is calculated based on maximum pressure acting on each area in which base plate is divided, neglecting non-uniform pressure from foundation on base plate on conservative side.

509.2.2.9. Elements of base plate main and secondary stiffeners are designed as per Clauses 509.2.2.4 and 509.2.2.4.1.

509.3. **Cap Plate**

Cap plate serves as medium for transferring the axial load from structure above (beam, girder) uniformly to the member/stanchion

509.3.1. The thickness of cap plate should be preferably 16-25 mm and stiffener's thickness should not be less than

$\frac{1}{15}\sqrt{\frac{2600}{F_y}}$ times width of stiffener, where F_y is the yield stress of stiffener in kg/cm^2

509.3.1.2. When the load from beam is transferred to stanchion member through bearing stiffeners extended beyond the beam, the cap plate serves as media to transmit this load to the stiffeners connected to web of stanchion/ member, or to tie beam for lattice member by rivet or weld. The cap plate is designed for specified load.

509.3.1.2.1. If the beam/girder is supported on stanchion in such a manner that loads are directly transmitted to the flange of stanchion or main element or lattice member, cap plate should be provided as per Clause 509.3.1 without calculation

509.3.1.3. The width of cap stiffeners is determined from bearing criteria $b_{sc} = \frac{N}{\sigma_{bg}.tc}$ and also shear stress should not

exceed specified stress : $\frac{N}{l_{sc}.t_c} \leq \sigma_s$

where,

l_{sc} = Length stiffener, to be sufficient for transmitting the load to web of stanchion by rivet or weld.

σ_{bg} = Basic permissible bearing stress

N = Load to be transmitted from girder/beam

b_{sc} = Width of stiffener

t_{c} = Thickness of the stiffener.

510. DESIGN OF TENSION MEMBERS

510.1. Design of tension member should generally follow the considerations under Clause 511.4 under "Trusses or Open Web Girders" of this Code.

511. DESIGN OF TRUSSES OR OPEN WEB GIRDERS

511.1. General

Trusses or open web girders are defined as triangulated skeletal girders.

For analysis of trusses, the following assumption may be made unless rigorous rigid frame analysis is adopted :

(a) All members are frictionless pin jointed.

(b) All members are straight and free to rotate at the joints.

(c) All loads including self weight of members are applied at the joints.

Stipulations made in this section are not applicable for design of stiffening trusses of suspension bridges.

511.2. Intersection at Joints

For triangulated trusses designed on the assumption of frictionless pin jointed connections, members meeting at a joint should, where practicable, have their centroidal axis meeting at a point,

and wherever practicable the centre of resistance of a connection shall lie on the line of action of the load so as to avoid an eccentricity moment on the connections.

If, at a joint, the centroidal axis of the adjacent members do not meet at a single point, the resulting flexural stresses in the members should be taken into account as secondary stress.

Where loads are not applied at truss joints, account should be taken of the following :

(a) resulting stresses where load is applied to a member in the plane of a truss other than at a joint.

(b) torsion and lateral flexure effects when the applied load is not in the plane of the truss. Where the load is applied to a cross member, the effect of interaction between the cross-member so loaded and the truss and adjacent cross member should be taken into account.

511.3. **Compression Members**

511.3.1. **General requirements:** This Clause covers the design of straight members of uniform cross-section subjected to axial compression or to combined compression and bending.

511.3.2. **Effective Section**

511.3.2.1. The properties of the cross-section should be computed from the effective sectional area. Where plates are provided solely for the purposes of lacing or battening these shall be ignored in computing the radius of gyration of the section.

511.3.2.2. The effective sectional area shall be the gross area less the specified deduction for excessive widths of plate (see Clauses 511.3.2.3 & 511.3.2.4) and the maximum deduction for open holes.

including holes for pins and black bolts (see relevant clause of this code) occurring in a section perpendicular to the axis of the member within the critical zone of the compression member.

511.3.2.3. For members other than circular hollow section for calculating the effective cross-sectional area of a member in compression the effective width b_e of a plate, in terms of its width b measured between adjacent lines of rivets, bolts or welds connecting it to other parts of the section, unless effectively stiffened shall be taken as :

(i) For riveted, bolted or stress relieved welded members in mild steel

For b/t not above 45, $b_e = b$

For b/t above 45, $b_e = 45t$

with a maximum value of $b/t = 90$

(ii) For riveted or bolted members in high tensile steel

For b/t not above 45, $b_e = b$

For b/t above 45, $b_e = 40t$

with a maximum value of $b/t = 80$

(iii) For welded members (not stress relieved) in mild steel or in high tensile steel

For b/t not above 30, $b_e = b$

For b/t above 30, $b_e = 40t \times [(b/t - 18) / (b/t - 14)]$

with a maximum value $b/t = 80$

In the above, "t" is the thickness of a single plate or the aggregate thickness of two or more plate, provided these are adequately tacked together considering maximum allowable pitch and edge distance of rivets or bolts.

511.3.2.4. The unsupported projection of any plate, measured from its edge to the line of rivets, bolts or welds connecting the plate to other parts of the section shall not exceed :

(i) 16t for Mild Steel

(ii) 14t for HT Steel

Where, "t" is as defined above. However, in case of compression flanges 't' is the thickness of the thinnest flange plate or the aggregate thickness of two or more plates when the projecting portions of these plates are adequately tacked together.

511.3.3. **Permissible Stress and Slenderness Ratio**

511.3.3.1. **Permissible stress :** Values of permissible stress in axial compression in MPa for some of the structural steels corresponding to various slenderness ratios are given in Table 11.1.

Table 11.1. Permissible Stress σ_{ac} (MPa) in Axial Compression for Steels with various Yield Stresses
(Clause 511.3.3.1)

	Yield stress (f_y) MPa		
$\lambda = l / r$↓	250	340	400
10	150	204	239
20	148	201	235
30	145	194	225
40	139	183	210
50	132	168	190
60	122	152	168
70	112	135	147
80	101	118	127
90	90	103	109
100	80	90	94

110	72	79	82
120	64	69	71
130	57	61	62
140	51	54	55
150	45	48	49
160	41	43	43
170	37	38	39
180	33	34	35
190	30	31	31
200	28	28	28
210	25	26	26
220	23	24	24
230	21	22	22
240	20	20	20
250	18	18	19

where, l = effective length of the member and r = radius of gyration

511.3.3.2. The ratio of the effective length to the least radius of gyration shall not exceed :

120 for main members, and

140 for wind bracings and subsidiary members.

511.3.3.3. All values of Permissible Stress in Axial Compression in MPa for Structural Steel with Yield Stress other than those shown in Table 11.1 may be calculated by using the following formula subject to the condition that s_{ac} shall not exceed $0.6\, f$

$$\sigma_{a} = \frac{0.6 f_{cc} x f_{y}}{[(f_{cc})_{n} P(f_{y})_{n}]^{1/n}}$$

where,

σ_{ac} = *permissible stress in axial compression, in MPa*
f_{y} = *yield stress of steel, in MPa*
f_{cc} = *elastic critical stress in compression,* = $\frac{\pi^2 E}{\lambda^2}$
E = *modulus of elasticity of steel; 2 x 10^5 MPa;*
$\lambda(=l/r)$ = *a slenderness ratio of the member, ratio of the effective length to appropriate radius of gyration; and*
n = *a factor assumed as 1.4*

511.3.4. **Lacing and Battening:** The open sides of built up compression members of channel or beam sections shall be connected by lacing or battening where the length of the outstand towards the open side exceeds 16 times the mean thickness of the outstand for mild steel and 14 times the mean thickness of the outstand for H.T. Steel.

511.3.5. Lacing and battening plates shall be designed in accordance with Clauses 511.3.9 and 511.3.10 and shall be proportioned to resist a total transverse shear force *Q* at any point in the length of the member equal to at least 2.5 per cent of the axial force in the member together with all shear due to external forces, if any, in the plane of lacing. The shear force *Q* shall be considered as divided equally among all transverse system and plating in parallel planes.

511.3.6. Compression members composed of two or more components connected as Described in Clauses 511.3.8, 511.3.9 and 511.3.10 may be designed as homogeneous members.

511.3.7. **Effective length of compression members other than lacings**

511.3.7.1. In riveted, bolted or welded trusses, the compression members act in a complex manner and the effective length to be used in computing allowable working stresses for compression members

shall be taken as given in Table 11.2 except that, for battened struts, all values given in table shall be increased by 10 per cent.

Table 11.2. Effective Lengh of Compression Members

Member		Effective length l of member		
			For buckling normal to the plane of the truss	
		For buckling in the plane of the truss	Compression chord or (compression) member effectively braced by lateral system	Compression chord or (compression) member unbraced
Chords		0.85 x distance between centres of intersection with the web members	0.85 x distance between centres of intersection with the lateral bracing members or rigidly connected cross girders	See Clause 511.3.7.4
Web	Single triangulated system	0.70 x distance between centres of intersection with the main chords	0.85 x distance between centres of intersections	Distance between centres of intersection
	Multiple intersection system where adequate connections are provided at all points of intersection	0.85 x greatest distance between centres of any two adjacent intersection	0.70 x distance between centres of intersection with the main chords	0.85x distance between centres of intersection with the main chords

Note :- The intersection referred to are those of the centroidal axis of the members.

511.3.7.2. For single angle discontinuous struts connected to gussets or to a section either by riveting or bolting by not less than two rivets or bolts in line along the angle at each end, or by their equivalent in welding, the eccentricity of the connection with respect to the centroid of the strut may be ignored and the strut designed as an axially loaded member provided that the calculated average stress does not exceed the allowable stresses given in Table 11.1 of Clause 511.3.3 in which "l" is the length of the strut, centre to centre of fastenings at each end and 'r' is the minimum radius of gyration.

511.3.7.3. For single angle discontinuous struts intersected by, and effectively connected to another angle in cross bracing, the effective length in the plane of bracings shall be taken as in Table 11.2 in Clause 511.3.7.1 and normal to the plane of bracing the effective lengths shall be taken as the distance along the bracing members between the points of intersection and the centroids of the main member. In calculating the ratio of slenderness, the radius of gyration about the appropriate rectangular axis shall be taken for buckling normal to the plane of the bracing and the least radius of gyration for buckling in the plane of the bracing.

511.3.7.4. **Effective length of unbraced compression chords:** For simply supported trusses with ends restrained at the bearings against torsion, the effective length l of the compression chord for buckling normal to the plane of the truss shall be taken as follows:

(a) With no lateral support to compression chord; where there is no lateral bracing between compression chords and no cross frames:

l = span

(b) With compression chords supported by U-frames, where there is no lateral bracing of the compression chord but where cross members and verticals forming U-frames provide lateral restraints:

$$l \quad = \quad 2.5 \times \sqrt[4]{(E\,I\,a\,\delta)} \text{ but not less than "}a\text{"}$$

where,

E = Young's modulus

I = maximum moment of inertia of compression chord about the axis lying in the plane of the truss.

a = distance between frames, and

δ = the virtual lateral displacement of the compression chord at the frame nearest to mid span of the truss, taken as the horizontal deflection of the vertical members. This deflection shall be computed assuming that the cross member is free to deflect vertically and that the tangent to the deflection curve at the centre of its span remains parallel to the neutral axis of the unrestrained cross member.

when δ is not greater than $\frac{a^3}{40EI}$

$l = a$

In case of symmetrical U-frames, where cross member and verticals are each of constant moment of inertia throughout their own length; it may be assumed that :

$$\delta \quad = \quad \frac{(d)^3 + (d_2)C}{3EI_1, EI_2}$$

where, d_1 = distance of the centroid of the compression chord from the top of the cross member

d_2 = distance of the centroid of the compression chord from the neutral axis of the cross member

C = half the distance between centres of the main trusses

E = Young's Modulus

I_1 = moment of inertia of the vertical in its plane of bending and

I_2 = moment of inertia of the cross member in its plane of bending

U-frames shall have rigid connections and shall be designed to resist, in addition to the effects of wind and other applied forces, the effects of a horizontal force F acting normal to the compression chord of the truss at the level of the centroid of this chord where:

$$F = \frac{1.4x100-3l}{\delta\left(\frac{Co}{}-1.7\right)}$$

In the above formula :

$$l = 2.5\ x\ \sqrt[4]{(E\ I\ a\ \delta\ )}$$

δ = the deflection of the chord under the action of unit horizontal force as difined above

C_o = Euler critical stress in chord = $\frac{\pi^2 E}{(l/r)2}$

where, E = Young's Modulus

r = radius of gyration

f_a = calculated working stress in the chord.

(c) With compression chord supporting continuous deck, a compression chord continuously supporting a reinforced concrete or steel deck shall be deemed to be effectively restrained laterally throughout its length (e.g., l = 0) if the friction or positive connection of the deck to the chord is capable of resisting a lateral force, distributed uniformly along its length of 2.5 per cent of the maximum force in the chord, in addition to other lateral forces.

511.3.8. Compression member composed of two components back to back.

511.3.8.1. Compression members composed of two angles, channels or tees back-to-back and separated by a distance not exceeding 50 mm shall be connected together by riveting, bolting or welding, so that maximum ratio of the slenderness l/r of each component of the member between such connections is not greater than 50 or 0.5 times the maximum ratio of slenderness of the member as a whole, whichever is less, where l is the distance between the centres of connection.

The number of connections shall be such that the member is divided into not less than three approximately equal parts.

511.3.8.2. Where the members are separated back-to-back the rivets or bolts in these connections shall pass through solid washers or packings, and where the connected angles, legs or tables of tees are 125 mm wide or over or where webs of channels are 150 mm wide or over, not less than two rivets or bolts shall be used in each connection, one on the line of each gauge mark.

511.3.8.3. Where these connections are made by welding, solid packings shall be used to effect the jointing unless the members are sufficiently close together to permit butt welding, and the members shall be connected by welding along both pairs of edges of the main components.

511.3.8.4. The rivets, bolts or welds in these connections shall be sufficient to carry the shear forces and the moments specified for battened struts and in no case shall the rivets or bolts be less than 16 mm.

511.3.8.5. Compression members connected by such riveting, bolting or welding shall not be subjected to transverse loading in a plane perpendicular to the riveted, bolted or welded surfaces.

511.3.8.6. Where components are in contact back-to-back riveting, bolting or intermittent welding shall be done in accordance with applicable clauses.

511.3.9. **Design of lacing of compression members**

511.3.9.1. As far as practicable, the lacing system shall not be varied throughout the length of the compression member.

511.3.9.2. Lacing bars shall be inclined at an angle of 40 to 70 degrees to the axis of the member where a single intersection system is used. and at an angle of 40 to 50 degrees where a double intersection system is used.

511.3.9.3. Except for tie plates as specified in Clause 511.3.9.8, double intersection lacing systems shall not be combined with members of diaphragms perpendicular to the longitudinal axis of the main member, unless all forces resulting from deformation are calculated and provided for in the lacing and its fastenings.

511.3.9.4. Lacing bars shall be so connected that there is no appreciable interruption of the triangulation of the system.

511.3.9.5. The maximum spacing of lacing bars whether by welding, riveting or bolting shall be such that the maximum slenderness ratio l/r of the components of the compression member between consecutive connections of the lacing bars to one component is not greater than 50 or 0.7 times the maximum ratio of slenderness of the member as a whole whichever is lesser where l is the distance between the centres of the connections of the lacing bars to one component.

511.3.9.6. The required section of lacing bars shall be determined by using permissible stresses for compression and tension members given in Clause 511.3.3.1 and Table 11.1. The ratio l/r of the lacing bars shall not exceed 140. For this purpose, the effective length l shall be taken as follows :

(a) **In riveted or bolted construction :** The length between the inner ends of rivets or bolts of the lacing bar in single intersection lacing and 0.7 times this length for double intersection lacing effectively connected at intersections.

(b) **In welded construction :** The distance between the inner ends of effective lengths of welds connecting the bars to the components in single intersection lacings and 0.7 times this length for double intersection lacing effectively connected at intersections.

511.3.9.7. Lacing bars shall be connected to the main members either by riveting or bolting by one or more rivets or bolts. in line along the lacings or by welding at each end sufficient to transmit the load to the bars. Any eccentricity of the connection with respect to the centroid of the lacing bar may be ignored and the lacing designed as an axially loaded strut provided that the calculated average stress does not exceed the allowable stress given in Table 11.1 or s_c. Where welded lacing bars over lap the main component, the amount of lap shall not be less than four times the thickness of the bar or four times the mean thickness of the flange of the component to which the bars are attached, whichever is less . Welding shall be provided at least along each side

of the bar for the full length of the lap and returned along the ends of the bar for a length equal to at least four times the thickness of the bar or width of the bar whichever is less.

Where lacing bars are fitted between the main components they shall be connected to each component by fillet welds on both sides of the bar or by full penetration butt welds.

511.3.9.8 Laced compression member shall be provided with tie plate at the ends of the lacing systems, at points where the lacing systems are interrupted and where the member is connected to another member.

511.3.9.9. The length of end tie plates measured between end fastenings along the longitudinal axis of the member shall not be less than (a) the perpendicular distance between the lines of rivets connecting them to the flanges or (b) the distance between vertical side plates of the main chords whichever is greater and shall be at least equal to (c) the depth of the cross girders where these are directly attached to the struts and the length of intermediate tie plates shall be not less than 3/4 of (a) above.

511.3.9.10. The thickness of tie plates shall not be less than 1/50 of the distance between the innermost lines of rivets, bolts or welds except when effectively stiffened at the free edges in which case the minimum thickness may be 8 mm. For this purpose the edge stiffener shall have a slenderness ratio not greater than 170.

511.3.9.11. Tie plates and their fastenings (calculated in accordance with the method described for battens) shall be capable of carrying the forces for which the lacing system is designed.

511.3.9.12. When angles, channels, etc. are used instead of end tie plates or are provided where the lacing system is interrupted, these shall be designed by the same method as for battens. The end batten or the intermediate batten and its fastenings shall be capable of carrying the forces for which the lacing has been designed. The l/r shall not exceed 140.

511.3.10. **Battening of compression members:** Battened compression members shall comply with the following requirements:

511.3.10.1. The battens shall be placed opposite each other at each end of the member and at points where the member stayed in its length and shall, as far as practicable, be spaced and proportioned uniformly throughout. The number of battens shall be such that the member is divided into not less than three bays within its actual length between centre-to-centre of connections.

511.3.10.2. In battened compression members when the slenderness ratio about the *Y-Y* axis (axis perpendicular to the battens) is not more than 0.8 times the ratio about the *X-X* axis, the spacing of battens between centre-to-centre of end fastenings shall be such that the ratio of slenderness *l/r* of the lesser main component over this distance shall not be greater than 50 or 0.7 times the ratio of slenderness of the member as a whole about its *X-X* axis (axis parallel to the battens).

In battened compression members in when the slenderness ratio about the *Y-Y* axis is more than 0.8 times the ratio about the *X-X* axis, the spacing of battens between centre -to-centre of end fastenings shall be such that the ratio of slenderness *l/r* of the lesser main component over this distance shall not be greater than 50 or 0.7

times the ratio of the slenderness of the member as a whole about its weaker axis.

511.3.10.3. Battens shall be plates, channels or *I* sections and shall be riveted, bolted or welded to the main components. Battens and their connections shall be so designed that they resist simultaneously a longitudinal shear force equal to QD/na and a moment equal to QD/2n where

D = the longitudinal distance between centre-to-centre of battens.

a = the minimum transverse distance between the centroids of rivet or bolt groups or welding.

Q = the transverse shear force as defined in Clause 511.3.5

n = the number of parallel planes of battens

511.3.10.4. The effective length of a batten parallel to the axis of a member shall be taken as the longitudinal distance between the end fastenings. End battens shall have an effective length of not less than (a) the perpendicular distance between the lines of rivets connecting them to the components, or (b) the distance between the vertical side plates of the main chords whichever is greater and shall be at least equal to (c) the depth of the cross girders where these are directly attached to the struts; and intermediate battens shall have an effective length of not less than 3/4 of (a) above, but in no case shall the length (of any batten) be less than twice the width of the smaller component in the plane of the battens.

511.3.10.5. The thickness of batten plates shall not be less than 1/50 of the minimum distance between the innermost lines of connecting rivet, bolts or welds, except when effectively stiffened at the free edges, in which case the minimum thickness may be 8 mm;

for the purpose the edge stiffeners shall have a slenderness ratio not greater than 170.

511.3.10.6. The length of weld connecting each longitudinal edge of the batten plate to a component shall in the aggregate be not less than half the length of the batten plate and at least one third of the weld shall be placed at each end of the longitudinal edge. In addition, the welding shall be returned along the ends of the plate for a length equal to at least four times the thickness of the plate.

Where tie or batten plates are fitted between main components they shall be connected to each component either by fillet welds on each side of the plate, at least equal in length to that specified in the preceeding paragraph or by complete penetration butt welds along the whole length of the plate.

511.3.10.7. Battened compression members not complying with these requirements, or those subjected to bending moments in the plane of the battens shall be designed according to the theory of elastic stability, or empirically with verification by tests, so that they have a load factor of not less than 1.7.

511.3.10.8. Battened compression member composed of two angles forming a cruciform cross-section shall conform to the above requirements except as follows :

(i) the battens shall be in pairs placed in contact one against the other, unless these are welded to form cruciform battens.

(ii) a transverse shear force $\frac{Q23}{\sqrt{2}}$ shall be taken as occurring separately about each rectangular axis of the whole member.

(iii) a longitudinal shear force of $\frac{QD}{a\sqrt{2}}$ and the moment shall be $\frac{QD}{2\sqrt{2}}$ taken in respect of each batten in each of

the two planes, except where the maximum value l/r can occur about a rectangular axis, in which case each batten shall be designed to resist a shear force of 2.5 per cent of the total axial force.

Note : Q, D and a as given in above formula are as defined in Clause 511.3.10.3

511.4. **Tension Members**

511.4.1. **General requirements:** Tension members should preferably be of solid cross-section. However, when composed of two or more components these shall be connected as described in Clauses 511.4.6, 511.4.7 and 511.4.8.

511.4.2. **Effective sectional area:** The properties of the cross section shall be computed from the effective sectional areas as given in Clause 507.8.2. When plates are provided solely for the purposes of lacing or battening, they shall be ignored in computing the radius of gyration of the section.

511.4.3. **Slenderness ratio:** For main members, the ratio of unsupported length to the least radius of gyration shall not exceed 300.

511.4.4. **Lacing and battening:** The open sides of built up compression members of channel or beam sections shall be connected by lacing or battening where the length of the outstand towards the open side exceeds 16 times the mean thickness of the outstand for mild steel and 14 times the mean thickness of the outstand for H.T. Steel.

511.4.5. Lacing and battening shall be designed in accordance with Clauses 511.4.7 and 511.4.8 and shall be proportioned to resist all shear forces due to external forces, if any, in the plane of lacing. The shear shall be considered as divided equally among all transverse systems and plating in parallel planes.

511.4.6. **Tension members composed of two components back-to-back**

511.4.6.1. Tension members formed by sections placed back-to-back, either in contact or separated by a distance not exceeding 50 mm shall be connected together in their length at regular intervals by riveting, bolting or welding so spaced that the maximum ratio of slenderness of each element is not greater than that specified for main members in Clause 511.4.3.

511.4.6.2. Where the components are in contact back-to-back riveting, bolting or welding shall be in accordance with Clauses applicable.

511.4.6.3. When the components are separated they shall be connected through solid washers or packings, riveted, bolted or welded.

511.4.7. **Design of lacing of tension members**

511.4.7.1. As far as practicable the lacing system shall not be varied throughout the length of the tension member.

511.4.7.2. Lacing bars shall be inclined at an angle of 40 to 70 degrees to the axis of the member when a single intersection system is used and at an angle of 40 to 50 degrees when a double intersection system is used.

511.4.7.3. Except for tie as specified in Clause 511.4.7.7 double intersection lacing systems shall not be combined with members or diaphragms perpendicular to the longitudinal axis of the member, unless all forces resulting from deformation of the member are calculated and provided for in the lacing and its fastenings.

511.4.7.4. Lacing bars shall be so connected that there is no appreciable interruption of the triangulation of the system.

511.4.7.5. The required section of the lacing bars shall be determined by using the permissible stresses for compression and tension members given in Clause 511.3.3 and σ_c. The ratio l/r of the lacing shall not exceed 170. For this purpose, the effective length shall be

taken as follows :

(i) In riveted or bolted construction, the length between the inner end rivets or bolts of the lacing bar in single intersection lacing and 0.7 times this length for double intersection lacing effectively connected at intersection.

(ii) In welded construction, the distance between the inner ends of effective lengths of welds connecting the bars to the components for single intersection lacing and 0.7 times this length for double intersection lacing effectively connected at intersection.

511.4.7.6. The riveting, bolting or welding of lacing bars to the main members shall be sufficient to transmit the load to the bars. Where welded lacing bars overlap the main components, the amount of lap shall not be less than four times the thickness of the bar or four times the mean thickness of the flange of the component to which the bars are attached, whichever is less. The welding shall be provided along each side of the bar for the full length of the lap and returned along the ends of the bar for a length equal to at least four times the thickness of the bar or width of the bar whichever is less.

Where lacing bars are fitted between main components, they shall be connected to each component by fillet welds on both sides of the bar or by full penetration butt welds.

511.4.7.7. Laced tension members shall be provided with tie plates at the ends of the lacing systems, at points where the lacing systems are interrupted and where the member is connected to another member.

511.4.7.8. The length of end tie plate parallel to the axis of the member shall not be less than (a) the perpendicular distance between the centroids of the main components and shall be at least equal to (b) the depth of the cross girders where these are directly attached to the member, and the length of the intermediate tie plates shall not be less

than 3/4 of (a) above.

511.4.7.9. The thickness of all tie plates shall not be less than 1/60 of the distance between the innermost lines of rivets, bolts or welds attaching them to the main components, except when effectively stiffened at the edges, in which case the minimum thickness may be 8 mm; for this purpose the edge stiffeners shall have a slenderness ratio not less than 170.

511.4.7.10. When angles, channels, etc. are used instead of end tie plates or are provided where the lacing system is interrupted, these shall be designed by the same method as for battens. The end batten or the intermediate batten and its fastenings shall be capable of carrying the forces for which the lacing has been designed. The l/r shall not exceed 140.

511.4.8. **Battening of tension members:** Battened tension members shall comply with the following requirements :

511.4.8.1. The spacing of battens, measured as the distance between the centres of adjacent end pitches of rivets or bolts or, for welded construction, the clear distance between the battens, shall be such that the maximum ratio of slenderness of each element is not greater than that specified for main members in Clause 511.4.3.

511.4.8.2. The effective length of the batten, parallel to the axis of the member, shall be taken as the longitudinal distance between end fastenings.

End battens shall have an effective length of not less than (a) the perpendicular distance between centroids of the main components and shall be at least equal to (b) the depth of the cross girders where these are directly attached to the members; and the length of the intermediate battens shall have an effective length of not less than one-half of (a) above.

511.4.8.3. Batten plates shall have a thickness of not less than 1/60 of the minimum distance between the connecting rivet or bolts groups or welds except where they are stiffened at their edges.

511.4.8.4. Where battens are attached by rivets or bolts, not less than two rivets or bolts shall be used in each connection. Where battens are attached by welds, the length of welds connecting each longitudinal edge of the batten plate to the component shall, in the aggregate, be not less than half the length of the batten plate, and at least 1/3 of the weld shall be placed at each end of the longitudinal edge. In addition, welding shall be returned along the ends of the plate for a length at least four times the thickness of the plate.

Where the tie or batten plates are fitted between main components they shall be connected to each member either by fillet welds on each side of the plate, at least equal in length to that specified in the preceding paragraph or by full penetration butt weld.

511.5. **Splices**

511.5.1. **For compression member:** Splices in compression members located at or near effectively braced panel points shall be designed to transmit the full design load in the member. All other splices in compression members shall have a sectional area 5 per cent more than that required to develop the load in the member at the average working stress of the member. All cover material shall, as far as practical, be so disposed with respect to the cross-section of the member so as to transmit the proportional load of the respective parts of the section.

511.5.1.1. Wherever possible both surfaces of the parts spliced shall be covered or other means taken to maintain the alignment of the abutting ends.

511.5.1.2. Where flexural tension may occur in the member, the cover material shall be designed to resist such tension.

511.5.1.3. Rivets, bolts or welds shall develop the full load in the cover material as defined above calculated on the cover area.

511.5.2. **For tension members**

511.5.2.1. Splices in tension members shall have a sectional area 5 per cent more than that required to develop the load in the member and, whenever practicable, the cover material shall be disposed to suit the distribution of stress in the various parts of the cross-section of the member. Both surfaces of the parts splices shall be covered wherever possible.

511.5.2.2. Rivets, bolts or welds shall develop the full load in the cover material as defined above, calculated on the cover area.

511.6. **Connections at Intersection**

511.6.1. Connections of members at an intersection shall develop at least the design loads and moments transmitted by the members. Due regard to the nature and distribution of stress over the cross-section of the members shall be given in determining the distribution of the fastenings. All members shall, where possible, be so connected that the load is appropriately distributed over the cross-section; otherwise, consideration shall be given to the distribution of stress through the material to those parts of the section not directly connected, and for this purpose the angle of distribution may be taken as 45°.

511.6.2. Gusset shall be capable of sustaining the design loads and moments transmitted by the members without exceeding the allowable working stresses.

511.6.3. Gusset plates shall be so shaped, and connectors so arranged as to avoid severe stress concentrations.

511.6.4. Rivet, bolt and welding groups shall be as compact as practicable.

511.7. **Lug Angles**

511.7.1. Lug angles connecting a channel or similar member shall, as far as possible, be disposed symmetrically with respect to the section of the member.

511.7.2. In the case of angle members the lug angles, and their connection to the gusset or other supporting member, shall be capable of developing a strength not less than 20 per cent in excess of the force in the outstanding leg of the angle and the attachment of the lug angles to the angle member shall be capable of developing a strength 40 per cent in excess of that force.

511.7.3. In the case of channel or similar members, the lug angles, and their connection to the gusset or other supporting member, shall be capable of developing a strength not less than 10 per cent in excess of the force not accounted for by the direct connection of the member and the attachment of the lug angles to the member shall be capable of developing a strength 20 per cent in excess of that force.

511.7.4. In no case shall less than two bolts or rivets be used for attaching the lug angle to the gusset or other supporting member.

511.7.5. The effective connection of the lug angle shall, as far as possible, terminate at the end of the member connected, and the fastening of the lug angle to the member shall preferably start in advance of the direct connection of the member to the gusset, etc.

511.8. **Section at Pin Holes in Tension Members**

In pin-connected tension members (generally used for erection purpose) the longitudinal net section beyond the pin hole parallel with the axis of the member shall be not less than the required net section of the member. The net section through the pin hole transverse to the axis of the member shall be at least 33 per cent greater than the required net section of the member. In the case of members without stiffened edges

the ratio of the net width of the members (through the pin hole transverse to the axis of the member) to its thickness shall not be more than 16. Where the thickness of the main material is not sufficient to resist the load from the pin in bearing, or where the net section through the pinhole requires reinforcement, pin plates (see Clause 511.9) shall be provided and the total thickness shall comply with the above requirements.

511.9. **Pin Plates**

Pin plates shall be of sufficient thickness to make up the required bearing or cross-sectional area and shall be so arranged as to reduce the eccentricity to a minimum. Their length measured from the centre of the pin to the end (on the reaction side) shall be at least equal to their width and at least one plate on each side shall be as wide as the dimensions of the member will allow. Pin plates shall be connected with enough rivets, bolts or welds to transmit the bearing pressure on them and shall be so arranged as to distribute it uniformly over the full section of the member.

511.10. **Diaphragms in Members**

In addition to diaphragms required for the proper functioning of the structure, diaphragms shall be provided as necessary for fabrication, transport and erection.

511.11. **Lateral Bracings**

511.11.1. All girders shall be provided with a lateral bracing system extending from end-to-end of sufficient strength to transmit the effect of wind, seismic and centrifugal forces, if any to the bearings.

511.11.2. The bracing on the loaded chord shall be so designed as to transmit to the main girders the longitudinal loads due to tractive effort and/or braking effect in order to relieve the cross girders of horizontal bending stress.

511.11.3. Where the depth permits, lateral diagonal bracing's shall be fixed between the top chords of main girders of through span, of sufficient rigidity to maintain the chords in line and of sufficient strength to transmit the wind or seismic forces to the portal bracing between end posts.

The floor system may be taken as part of the bracing system provided it is designed for that purpose.

511.11.4. The lateral bracings between compression chords shall be designed to resist a transverse shear at any section equal to 2.5 per cent of the total compressive force carried by both the chords at the section under consideration. This force should be considered in addition to the wind, and centrifugal forces.

511.11.5. **Sway bracings:** Wherever the depth of girder allows, the intermediate cross bracings or sway bracings between vertical web members shall be proportioned to transmit to the chord supported on bearings through the web members at least 50 per cent of the panel lateral load and the vertical members shall be designed to resist the resulting bending moment. The sway bracing so provided shall not be taken as affording any relief to the lateral bracing system or portal system.

511.11.5.1. **Portal bracings :** Through truss spans shall be provided with suitably designed portal system, as deep as the clearance will allow. The portal system shall be designed to take the full end reaction of the top chord lateral system and the end posts of the portal shall be designed to transfer this reaction to the bearings. In addition, the portal system shall be designed to resist a lateral shear equal to $1^{1/4}$ per cent of the total compressive force in the end posts or in the top chords in the end panel whichever is greater.

512. CONNECTIONS

512.1. **Composite Connections**

In any connection which takes a force directly communicated to it and which is made with more than one type of fastening, only rivets and turned and fitted bolts may be considered as acting together to share the load. In all other connections sufficient number of one type of fastening shall be provided to communicate the entire load for which the connection is designed.

512.2. **Welded Connections**

512.2.1. **Types of welds:** The following types of welds can be used :

(a) Continuous full penetration or partial penetration butt welds

(b) Continuous or intermittent fillet welds

(c) Plug welds

Intermittent butt welds shall not be used.

Partial penetration butt welds shall not be used for transmitting tensile forces or bending moments along longitudinal axis of the welds. Plug welds shall not be used for transmitting loads or moments and shall be used only to prevent the buckling or separation of lapped parts or to joint components of built-up members.

512.2.2. **Strength of weld**

512.2.2.1. **Butt weld**

The strength of a full penetration butt weld shall be taken as equal to the strength of the weaker of the parts joined provided the yield stress of the weld metal is atleast equal to that of the parent metal.

The strength of a partial penetration butt weld together with its reinforcing fillet weld, if any, shall be calculated as for a full penetration

fillet weld. The throat thickness shall be taken as

(a) the depth of weld preparation where this is of the J or U type.

(b) the depth of weld preparation minus 3 mm where the preparation is the V or bevel type

512.2.2.2. **Fillet weld:** The strength of a fillet weld shall be based on the effective throat thickness and the effective length

The effective throat thickness shall be considered as the height of a triangle that can be inscribed within the weld and measured perpendicular to its outer side.

The effective length shall be considered as the actual length minus twice the leg length . In case of fillet welds with end returns as per Clause 512.2.3.1 the effective length shall be considered as the actual length.

512.2.3. **General requiring welds**

512.2.3.1. **Fillet welds:** Maximum leg length of a fillet weld shall be 1 mm less than the thickness of the connected parts at the edge.

Minimum leg length of a fillet weld shall be in accordance with Clause 502.1.8, IS:9595-1980.

Intermittent fillet welds: Intermittent welds shall not be used in tension region of any member, considering their weakness in fatigue. When used in other regions, the undernoted stipulations shall apply.

The clear unconnected gap between the ends of the welds whether in line or staggered shall not be more than 200 mm and also shall not be more than:

(a) 12 times the thickness of the thinner part when the part is in compression

(b) 16 times the thickness of the thinner part when the part is in tension

(c) One-quarter of the distance between stiffeners when used to connect stiffeners to a plate or other part subject to compression or shear

In a line of intermittent welds, there shall be a weld at each end of the part connected.

In built-up members in which plates are connected by intermittent welds, continuous side fillet welds shall be provided at the ends of each side of the plate for a length at least equal to three quarter of the width of the narrower plate concerned. In exceptional cases, where this is not possible, the intermittent plug or slot weld shall be provided to prevent separation.

End returns

The fillet weld shall be returned continuously around the corner at the end of the side of a part for a length beyond the corner of not less than twice the leg length of the weld.

End connections by side fillets

If side fillets alone are used in end connections, both sides of the part shall be welded and the length of the weld on each side shall not be less than the distance between the welds nor less than 4 times the thickness of the thinnest part connected. Where the distance between the welds exceeds 16 times the thickness of thinnest part connected, intermediate plug or slot welds shall be used to prevent separation.

End connections by transverse welds

The overlap between the connected parts shall not be less than four times thickness of the thinnest part and the parts shall be connected by two transverse lines of welds. Where the distance between the weld exceeds 16 times the thickness of the thinnest part connected intermediate slot or plug welds shall be used to prevent separation.

Welds with packings

Where two parts connected by welding are separated by packing having thickness less than the leg length of a weld necessary to transmit the force, the required leg length will be increased by thickness of the packing. The packing shall be trimmed flush with the edge of the part which is to be welded. Where two parts connected by welding are separated by packing having a thickness equal to or greater than the leg length of weld necessary to transmit force, each of the parts shall be connected to the packing by a weld capable of transmitting the design force.

Welds in holes and slots

Fillets welds in holes or slots may be used to transmit shear in lap joints or to prevent the buckling or separation of the lapped parts or to join components of built-up members.

512.2.3.2. **T Butt Joints:** Butt welds in T joints shall be completed by means of fillet welds each having a size of not less than 25 per cent of the thickness of the outstanding part.

512.2.3.3. **Plug welds:** The entire area of the hole or slot shall be filled with weld metal having a thickness:

(a) equal to the thickness of the holed or slotted part where it is 16 mm or less.

(b) In other cases, not less than any of the following :

(i) 16 MM

(ii) 0.45 times the diameter of the hole or the width of the slot.

(iii) One-tenth of the length of slot but not greater than the thickness of the holed or slotted part.

The diameter of the hole or the width of a slot shall not be less than the thickness of the hole or slotted parts plus 8 mm.

The distance between centres of holes or between the centre lines of slots shall not be less than four times the diameter of the hole or the width of the slot. The distance between the centres of the slots measured in the direction of their length shall not be less than double the length of the slot.

The ends of the slot shall be semi-circular except where the slot terminates at the edges of the part where it can be square.

Welding procedure

The welding procedure and details shall be in accordance with IS:9595-1980 unless otherwise stipulated in this chapter.

512.3. **Connections (other than welded)**

512.3.1. **General:** Connections and splices in all members shall be made by the use HSFG bolts, bolts, rivets or other acceptable fasteners. The arrangement of plates, rolled sections and other connecting elements shall be such as to make proper provision for all axial, flexural shear and/or torsional stresses in the members being connected.

Bolted or riveted splices in all compression members shall be located as near as practicable to points of effective lateral support.

A member carrying a calculated stress shall not have a splice or connection with a single rivet or bolt.

Connections and splices for minor members, such as light bracing members, railings etc. may be permitted to have single bolted or riveted connections.

Minimum dia of bolts and rivets used in load bearing members shall be 16 mm diameter.

512.3.2. **Connections and splices in flexural members**

(a) The connection between a flange and a web of a built-up girder shall be designed to transmit the longitudinal shear force in the flange combined with any vertical loads which are directly applied to the flanges.

(b) **Flange splices**

(i) **General:** Flange splices may be made to join flange components made from the same grade of steel but may be of different cross-sections.

(ii) **Bolted or riveted splices:** Where bolted or riveted splice plates are used to obtain a splice in a flange the sum of their areas shall be at least 5 per cent more than the area of the flanges as spliced. The centres of gravity of the sections on either side of the splice shall coincide as far as practicable. The splice connections on each side of the splice shall be capable of transmitting at least the greater of -

(1) 1.10 times the computed force in the flange at the splice point.

(2) 0.80 times the maximum safe force in the weaker flange, calculated from the basic allowable stress, the net section being used for tension flanges and the gross section for compression flanges.

(c) **Web splice:** A splice in the web of a plate girder or rolled section used as a beam shall be designed to resist the shearing forces and the portion of the design moment resisted by the web, and for the moment due to the eccentricity of shear introduced by the splice connection.

Web plates shall be spliced symmetrically by plates on each side. The splice plates shall extend as far as practicable for the full depth of the web. There shall not be less than at least two rows of rivets or bolts on each side of the joint.

512.3.3. **Connections in triangulated structures**

(a) **Eccentric connections:** Axially stressed members meeting at a joint shall have their gravity axes intersect at a point if practicable; if not, provision shall be made for bending stresses due to the eccentricity.

(b) **Connections at Interconnections:** Connection of members at an intersection shall develop at least 1.10 times the design loads and moments transmitted by the members. Due regard to the nature and distribution of stress over the cross-section of the members shall be given in determining the distribution of the fasteners.

All members shall, where possible, be so connected that the load is appropriately distributed over their cross-section.

If this is impracticable, consideration shall be given to the way in which the stresses at the joint are distributed to those parts of the cross-section of the member which are not directly connected at the joint. For this purpose the angle of distribution of stress may be taken as 45°.

Gusset plates shall be capable of sustaining 1.05 times the design loads and moments transmitted by the members. If an unsupported edge of a gusset plate is in compression and if the length of such edge exceeds

50 times the thickness of the gusset plate, the edge shall be suitably stiffened.

(c) **Splices in tension members and compression members of non bearing type:** Such splices shall be made symmetrical about the gravity axes of the members as far as is practicable.

Bolted riveted splices shall be designed for any applied moment and the greater of-

(i) 1.10 times the computed forces in the member, and

(ii) 0.80 times the safe load in the member calculated from the appropriate basic allowable stress.

The ends of the members need not be in close contact.

(d) **Splices in compression members of bearing type:** In bearing type splices in a compression member, the ends of the members shall be machined and assembled to be in close contact with each other. For a bearing splice it may be assumed that the machined faces transmit 50 per cent of the compressive force in the member. The splice plates and connection shall be however designed to transmit 60 per cent of the compressive force in the member and the applied moment, if any.

Note : Before specifying bearing splices the designer shall, however, satisfy himself that such facilities for machining are available in the particular project.

512.3.4. **Rivets and bolts**

(a) **Gross and net areas of rivets and bolts:** The gross area of a rivet shall, be taken as the cross-sections area of the rivet hole.

The net sectional area of a bolt or screwed tension rod shall be taken as the tension area for the particular diameter of bolt as given in the table below:

Nominal Thread Dia (mm)	12	14	16	18	20	22	24	27	30	33
Nominal Stress Area (mm^2)	84	115	157	192	245	303	353	459	561	694

(b) **Calculation of stresses:** Calculation of stresses in rivets and bolts shall be as per Clause 506.5.2.

(c) **Diameter of rivet and bolt holes:** The diameter holes of a rivet of upto 25 mm nominal diameter shall be taken as 1.5 mm larger than the nominal diameter of the rivet and 2.0 mm larger than the nominal rivet diameter in case of larger diameter rivets.

The diameter of a bolt hole shall be taken as the nominal diameter of the bolt plus 1.5 mm unless otherwise specified.

(d) **Edge distances:**

(i) In case of rolled, machine flame cut, sawn or plane edges the distance between the centre of the rivet or bolt hole to such edge shall not be less than 1.5 times the diameter of the hole.

(ii) In case of sheared or hand flame cut edges the edge distance shall be 1.75 times the diameter of the hole.

(e) **Pitch of rivets or bolts**

(i) The minimum distance between the centres of any two adjacent rivets or bolts shall not be less than 2.5 times the nominal diameter of the connector.

(ii) The maximum distance between the centres of any two adjacent rivets or bolts connecting members either in tension or in compression shall not exceed either 32t or 300 mm, where t is thickness of the thinner outside element.

(iii) The distance between centres of two adjacent rivets or bolts in a line along the direction of stress shall not exceed 16t or 200 mm in tension members, and 12t or 200 mm in compression members. In the case of compression members transferring forces through butting faces the pitch shall not exceed 4.5 times the diameter of rivet or bolt from the abutting faces. This pitch will apply for a distance equal to 1.5 times the width of the member.

(iv) When rivets bolts are staggered at equal intervals and the gauge does not exceed 75 mm, the distance as specified in (ii) and (iii) above between centre of adjacent connectors may be increased by 50 per cent.

(f) **Long rivets:** The grip of rivets carrying calculated loads shall not exceed 8 times the diameter of the holes. Where the grip exceeds 6 times the diameter of the hole, the number of rivets required by normal calculation shall be increased by not less than half a per cent for each additional millimeter of length of grip above 6 times the hole diameter.

(g) **Rivets with counter sunk head:** For counter sunk rivets, half of the depth of the counter sinking shall be neglected in calculating the length of the rivet in bearing. As far as possible rivets in tension shall be avoided. However, when rivets with contour sunk heads are in tension, the tensile value of the rivets shall be reduced by 33-1/3 per cent. No reduction need in shear.

(h) **Rivets or bolts through packing**

Number of rivets or bolts carrying calculated shear through a packing shall be increased above the number required by normal calculations by 2.5 per cent for each 2.0 mm thickness of packing except that, for packing having a thickness of upto 6 mm, no increase need be made. For double shear connections packed on both sides, the number of additional rivets or bolts required shall be determined from the thickness of the thicker packing. The additional rivets or bolts shall be placed in an extension of the packing.

(i) **Staggered pitch:** When rivets and bolts are staggered at equal intervals and the gauge does not exceed 75 mm, the distances between centres of rivets and bolts as specified earlier may be increased by 50 per cent.

513. FABRICATION AND INSPECTION

513.1. General

All work shall be in accordance with the drawings and clauses of this code unless otherwise agreed.

513.2. Laminations in Plates

The following areas of plate shall not have laminations exceeding the prescribed limits :

(a) Steel plate and sections in which tension stresses are transmitted through thickness of plate or in region in which lamination could affect the buckling behaviour under compression and bending compression.

(b) On each side of welded bearing diaphragm, strip of flange and web plate having width equal to 25 times of their thickness.

(c) The strip of web plate having 25 times thickness on each side of single sided bearing stiffener welded to web.

(d) For welded cruciform joints transmitting tensile stress through the plate thickness on strip having width four times the thickness of plate on each side of attachment.

(e) For edges of plates where corner welds are provided on to the surface of such plates.

Other areas of plate, section specified by the Engineer shall not have lamination exceeding prescribed limits.

513.3. Storage of Materials

All material, consumable, including raw steel or fabricated material shall be stored specificationwise and sizewise above the ground

upon platforms, skids or other supports. These shall be kept free from dirt and other foreign matter and shall be protected from corrosion and distortion. The electrodes shall be stored specificationwise and shall be kept in dry warm condition in properly designed racks. The bolts, nuts, washers and other fasteners shall be stored on racks above the ground with protective oil coating in gunny bags. The paint shall be stored under cover in air tight containers.

513.4. **Straightening, Bending and Pressing**

513.4.1. Straightening and flattening of steel shall be done by methods that will not injure the metal. Hammering shall not be permitted.

Straightening by heating shall be done under controlled procedure. Temperature of the steel shall not be more than 650°C or the temperature timing and cooling rate shall be appropriate to the particular type of steel and shall be agreed by the authorities. Accelerated cooling shall not be used without the approval of Engineer.

513.4.2. **Bending and curving**

513.4.2.1. Steel having yield stress more than 360 MPa shall not be heat curved.

513.4.2.2. Heating procedure - Rolled beams and girders may be curved by either continuous or V-type heating as approved by Engineer.

(a) For the continuous method, a strip of sufficient width along the edge of top and bottom flange shall be heated simultaneously to desired temperature to obtain required curve.

(b) For V-type of heating, the top and bottom flanges shall be heated in truncated triangular or wedge-shaped areas having their base along the flange edge and spaced at regular intervals along each flange. The truncated triangular pattern shall have an angle 15 to 30 degrees with base not more than 250 mm. The spacing and temperature shall be as

required to obtain the required curvature and heating shall be at approximately same rate along the top and bottom flange.

For flange thickness of 32 mm or more, both inside and outside surfaces shall be heated concurrently.

513.4.2.3. **Temperature:** The heat bending shall be conducted so that the temperature of steel does not exceed 620°C. The girder shall not be artificially cooled until temperature comes down to 315°C by natural cooling. The method of artificial cooling has to be approved by Engineer.

513.4.2.4. **Camber:** Camber for rolled beams may be obtained by heat curving methods approved by Engineer. For camber in plate girders, the web shall be cut to prescribed camber with suitable allowance for shrinkage due to cutting, welding and heat curving

513.5. **Workmanship**

513.5.1. Fabricator has to submit a Quality Assurance Plan according to the nature of fabrication work, such as, welded fabrication or riveted fabrication and the same should be approved by the client. Quality Assurance Plan should elaborate Nodal point checking and inspection during the stage of fabrication and also the materials.

513.5.2. Fabrication work shall be taken up only after receipt of approved fabrication/working drawing.

513.5.3. All members shall carry mark number and item number and, if required, serial number. Method of marking shall commensurate with the process of manufacture and shall ensure retention of identity at all stages.

513.5.4. **Preparation of edges ends and surfaces:** Material shall be cleaned and any burring, scales or abnormal irregularities shall be removed.

513.5.4.1. **Edge and end planing/cutting:** End/edge planing and cutting shall be done by any one of the following prescribed methods or left as rolled.

(a) Shearing, cropping, sawing, machining, machine flame cutting.

(b) Hand flame cutting with subsequent grinding to a smooth edge.

(c) Sheared edges of plate not more than 16 mm thick with subsequent grinding to smooth profile, which are for secondary use such as stiffeners and gussets.

If ends of stiffeners are required to be fitted they shall be ground so that the maximum gap over 60 per cent of the contact area does not exceed 0.25 mm.

513.5.4.2. **Shearing and flame cutting:** Where flame cutting or shearing is used as specified in Clause 513.4.1 atleast one of the following requirements shall be satisfied.

(a) The cut edge is not subjected to applied stress.

(b) The edge is incorporated in weld.

(c) The hardness of cut edge does not exceed 350 HV 30.

(d) The material is removed from edge to the extend of 2 mm or minimum necessary, so that hardness is less than 350 HV 30.

(e) Edge is suitably heat-treated by approved method to the satisfaction of Engineer and shown that cracks had not developed by dye penetrant or magnetic particle test.

(f) Thickness of plate is less than 40 mm for machine flame cutting for materials conforming to IS:2062. The requirement of hardness below 350 HV 30 of flame cut edges should be specified by Engineer. Wherever specified by the Engineer the flame cut edges shall be ground or machined over and above requirement (a) to (f).

513.5.4.3. Where machining for edge preparation in butt joint is specified, the ends shall be machined after the members have been fabricated.

513.5.4.4. Outside edges of plate and section, which are prone to corrosion shall be smoothened by grinding or filling.

513.5.5. **Rivet and bolt holes**

513.5.5.1. Holes for rivets, black bolts, high strength bolts and countersunk bolts/rivets (Excluding close tolerance and turn fitted bolts): All holes for rivets or bolts shall be either punched or drilled. The diameter of holes shall be 1.5 mm larger for bolts/rivets upto 25 mm dia and 2 mm for more than or equal to 25 mm.

All holes shall be drilled except for secondary members such as, floor plate, handrails etc., and members which do not carry the main load can be punched subject to the thickness of member does not exceed 12 mm for material conforming to IS:2062.

Holes through more than one thickness of material or when any of the main material thickness exceeds 20mm for steel to IS:2062 or 16 mm to IS:8500, IS:8500 shall either be subdrilled or subpunched, less than 3 mm diameter than required size and reamed to full diameter. The reaming of material more than one thickness shall be done after assembly.

513.5.5.2. Holes for close tolerance and turn fitted bolts. The diameter of the holes shall be equal to - 0.15 mm to - 0.0 mm, of the bolt shank.

The members to be connected with close tolerance or turn fitted bolts shall be firmly held together by service bolts or clamped and drilled through all thickness in one operation and subsequently reamed to required size within specified limit of accuracy as specified in IS:919 tolerance grade H8.

The holes not drilled through all thicknesses at one operation shall be drilled to smaller size and reamed after assembly.

513.5.5.3. Holes for high strength friction grip bolts - All holes shall be drilled after removal of burrs. Where the number of plies in the grip does not exceed three, the diameters of holes shall be 1.5 mm larger than those of bolts and for more than three plies in grip, the diameter of hole in outer plies shall be as above and diameter of holes in inner plies shall be not less than 1.5 mm and not more than 3 mm larger than those in bolts, unless otherwise specified by Engineer.

513.5.6. **Bolted construction**

513.5.6.1. All joint surface for bolted connection including bolts, nuts, washers, shall be free of scale, dirts, burrs other foreign material and other defects that would prevent solid seating of parts. The slope of surface of bolted parts in contact with bolt head and nuts shall not exceed 1/20, plane normal to bolts axis, otherwise suitable tapered washer shall be used.

All fasteners shall have a washer under nut or bolt head, whichever is turned in tightening.

Each fastener of joint shall be tightened to specified value or equal to 70 per cent of specified minimum tensile strength by hand wrenches (turn of nut method) or calibrated wrenches or manual torque wrenches. Impact wrench or any other method specified by Engineer.

When turn of nut method is used for tightening the bolts in joint first all bolts shall be brought to "snug tight" condition, that is tightening by full effort of man using ordinary wrench or by few impacts of any impact wrench. All bolts in the joint shall be then tightened additionally by applicable amount of nut rotation specified below for guidance :

Bolt length (from underside of head to edge)	Disposition of outer faces of bolted parts	
	Bolt face normal to bolt axis	One face Normal to bolt axis and other face slopped less than 1:20
Upto and including 4 dia	1/3 turn	1/2
Over 4 dia but less than 8 dia	1/2 turn	2/3
Over 8 dia but less than12dia	2/3 turn	5/6

513.5.6.2. **High strength friction grip bolts and bolted connections:** The general requirement shall be as per relevant IS specifications mentioned in Clause 505.3 (Fasteners) of this code. Unless otherwise specified by the Engineer, bolted connections of structural joints using high tensile friction grip bolts shall comply with requirements mentioned in IS:4000-1992.

513.5.6.3. **Riveted construction**

513.5.6.4. Assembled riveted joint surfaces including those adjacent to the rivet head shall be free of scale. dirt, loose scale, burrs, other foreign material and defects that would prevent solid seating of parts.

513.5.6.5. The part/members to be riveted shall be firmly drawn together with bolts, clamps or tack weld. Every third hole of the joint shall have assembly bolts till riveted. Drift shall be used only for matching of holes of the parts/members, but not to the extent as to distort the holes. Drift of larger size than the normal diameter of the holes shall not be used.

513.5.6.6. Rivets shall be heated uniformly to a "light cherry red colour" between 650°C to 700°C for hydraulic riveting and "Orange colour" for pneumatic riveting of mild steel rivets. High tensile steel rivets shall be heated upto 1100°C. Any rivet whose point is heated more than prescribed shall not be driven.

513.5.6.7. Rivet shall be driven in hole when hot so as to fill the hole as completely as possible and shall be of sufficient length to form a head of the standard dimension. When countersunk head is required the head shall fill the countersunk hole. Projection after countersinking shall be ground off wherever necessary.

513.5.6.8. The riveting shall be done by hydraulic or pneumatic machine unless otherwise specified by the Engineer.

513.5.6.9. Any defective rivet due to defect in head size or head driven off the centre shall be removed and replaced.

513.5.6.10. The parts not completely riveted in the shop shall be secured by bolts to prevent damage during transport and handling.

513.5.7. **Welded construction**

513.5.7.1. Surfaces and edges to be welded shall be smooth, uniform and free from fins, tears, cracks and other discontinuities. Surface shall also be free from loose or thick scale, slag rust, moisture oil and other foreign materials.

513.5.7.2. The general welding procedures including particulars of the preparation of fusion faces for metal arc welding shall be carried out in accordance with IS:9595.

513.5.7.3. The welding procedures for shop and site welds including edge preparation of fusion faces shall be submitted in writing in accordance with Clause 22 of IS:9595 for the approval of the Engineer before commencing fabrication, and shall also be as per details shown on drawings. Any deviation for above has to be approved by the Engineer.

513.5.7.4. Electrodes to be used for metal arc welding shall comply with relevant IS specifications mentioned in Clause 505.3 of this code. Procedure test shall be carried out as per IS:8613 to find out suitable wire-flux combination for welded joint.

513.5.7.5. Assembly of parts for welding shall be in accordance with Clauses 14 to 16 of IS:9595.

513.5.7.6. The welded temporary attachment should be avoided as far as possible, otherwise the method of making any temporary attachment shall be approved by Engineer. Any scars from temporary attachment shall be removed by cutting, chipping and surface shall be finished smooth by grinding to the satisfaction of Engineer.

513.5.7.7. For welding of any particular type of joint, welders shall qualify to the satisfaction of Engineer in accordance with appropriate welders qualification test as prescribed in any of the Indian standards IS:817-1966, IS:1393-1961, IS:7307 (Part-I)-1974, IS:7310 (Part-I)-1974 and IS:7318 (Part I)-1974 as relevant.

513.5.7.8. In assembling and joining parts of a structure or of built-up members, the procedure and sequence of welding shall be such as to avoid distortion and minimise shrinkage stress.

513.5.7.9. All requirements regarding preheating of present material and interpass temperature shall be in accordance with provisions of IS:9595.

513.5.7.10. Peening of weld shall be carried out wherever specified by the Engineer.

(a) If specified, peening may be employed to be effective on each weld layer except first.

(b) The peening should be carried out after weld has cooled by light blows from a power hammer, using a round nose tool.

Care shall be taken to prevent scaling or flecking of weld and base metal from over peening.

513.5.7.11. Where the Engineer has specified the butt welds are to be ground flush, the loss of parent metal shall not be greater than that allowed for minor surface defects.

513.5.7.12. The joints and welds listed in are prohibited type, which do not perform well under cyclic loading:

(a) Butt joints not fully welded throughout their cross section.

(b) Groove welds made from one side only without any backing strip.

(c) Intermittent groove welds.

(d) Intermittent fillet welds.

(e) Bevel-grooves and J-grooves in butt joints for other than horizontal position.

(f) Plug and slot welds.

513.5.7.13. The run-on and run-off plate, extension shall be used providing full throat thickness at the end of butt welded joints. These plates shall comply with following requirements.

(i) One pair of "run-on" and one pair of "run-off" plates prepared from same thickness and profile as the parent metal shall be attached to start and finish of all butt welds preferably by clamps.

(ii) When "run-on" and "run-off" plates shall be removed by flame cutting, it should be cut at more than 3 mm from parent metal and remaining metal shall be removed by grinding or by any other method approved by the Engineer.

513.5.7.14. **Welding of stud shear connectors:**

(a) The stud shear connectors shall be welded in accordance with the manufacturer's instructions including preheating.

(b) The stud and the surface to which studs are welded shall be free from scale, moisture, rust and other foreign material. The stud base shall not be painted, galvanised or cadmium-plated prior to welding.

(c) Welding shall not be carried out when temperature is below 0°C or surface is wet.

(d) The welds shall be visually free from cracks and lack of fusion and shall be capable of developing atleast the nominal ultimate strength of studs.

(e) The procedural trial for welding the stud shall be carried out when specified by the Engineer.

513.5.8. **Annealing and stress relieving :** The members which are indicated in the contract or specified by Engineer, to be annealed or stress relieved shall have finish machining, boring etc. done subsequent to heat treatment. The stress relief treatment shall conform to the following unless specified by the Engineer.

(a) The temperature of the furnace shall not be more than 300°C at the time welded assembly is placed in.

(b) The rate of heating shall not be more than 220°C per hour divided by max. metal thickness subject to max. 220°C per hour.

(c) After max. temperature of 600°C is reached, the assembly shall be held within specified limit of time based on weld thickness. The temperature shall be maintained uniformly throughout the furnace during holding period such that temperature at no two points on the member will differ by more than 80°C.

(d) The cooling shall be done in closed furnace when temperature is above 300°C at the max. rate of 260°C per hour divided by max. metal thickness. The local stress relieving shall be carried out if specified and procedure approved by the Engineer.

513.5.9. **Pins and pin holes:** The pins shall be of required length, parallel throughout and of smooth surface free from flaws. The pin holes shall be bored smooth, straight and true to gauge and right angles to the axis of the member. Boring shall be done only after member is finally riveted, bolted or welded unless otherwise approved by the Engineer. To facilitate insertion and extraction, pins may be chamfered beyond the required length and provided with suitable holes in the chamfered portion.

513.5.10. **Rectification of surface defects and edge laminations:** The surface defects revealed during fabrication or cleaning shall be repaired as specified. The repair by welding on any surface defect or exposed edge lamination shall be carried out only with approval of the Engineer.

513.5.11. **Shop assembly:** The steel work shall be temporarily assembled at place of fabrication. Assembly shall be full truss or girder, unless progressive truss or girder assembly, full chord assembly, progressive chord assembly or special, complete structure assembly is specified by the Engineer.

The field connections of main members of trusses, arches, continuous beam spans, bents, plate girders and rigid frame assembled, aligned, accuracy of holes, camber shall be checked by the Engineer and then only reaming of subsize holes to specified size shall be taken up.

The assembly will be dismantled after final drilling of holes and approval of the Engineer.

The camber diagram showing camber at each panel point, and method of shop assembly and any other relevant detail shall be submitted to the Engineer for approval.

513.5.12. **Fabrication tolerances**

513.5.12.1. In general all parts in an assembly shall fit together accurately within tolerances specified in Table 13.1, unless otherwise specified by the Engineer and agreed in contract.

Table 13.1. Fabrication Tolerances

(Clause 513.5.12)

Individual Components :

(1) Length :			
	(a)	Member with both ends finished for contact bearing	± 1 mm
	(b)	Individual components of members with end plate connection	+ 0 mm - 2 mm
	(c)	Other members -	
		(i) Upto and including 12 m	± 2 mm
		(ii) Over 12 m	± 3.5 mm
(2) Width :			
	(a)	Width of built-up girders	± 3 mm
	(b)	Deviation in the width of members required to be inserted in other members	0 mm - 3mm
(3) Depth :		Deviation in the depths of solid web and open web girders	3 mm -2 mm
(4) Straightness:			
	(a)	Deviation from straightness of columns to a maximum of 15 mm	L/300 subject to
		(i) In elevation	+ 5 mm
		(ii) In plan	- 0 mm
(5)		Deviation of centre line of web from centre line of flanges in built-up members at contact surfaces	3 mm
(6)		Deviation from flatness of plate webs of built-up members in a length equal to the depth of the member	0.005 d to a max. 2 mm
(7) Tilt of flange of plate girders			
	(a)	at splices and stiffeners, at supports, at the top flanges of crane girders, at bearings.	0.005 b to a max. of 2 mm

	(b) at other places	0.015 b to a maximum of 4 mm
(8)	Deviation from squareness of flange to web of columns and box girders	L/1000, where L is the nominal length of the diagonal
(9)	Deviation from squareness of fixed base plate (not machined) to axis of column. This dimension shall be measured parallel to the longitudinal axis of the column at points where the outer surfaces of the column sections make contact with the base plate.	D/500, where D is the distance from the column axis to the point under consideration on the base plate.
(10)	Deviation from squareness of machined ends to axis of columns	D/1000, where D is as defined in 9 above.
(11)	Deviation from squareness of machined ends to axis of beam or girders	D/1000, where D is as defined in 9 above.
(12)	Ends of members abutting at joints through cleats or end plates, permissible deviation from squareness of ends.	1/600 of depth of member subject to a max. of 1.5 mm.

513.5.12.2. A machined bearing surface, where specified by the Engineer, shall be machined within a deviation of 0.25 mm for surfaces that can be inscribed within a square of side 0.5 m.

513.5.13. **Alignment at splice and butt joints**

513.5.13.1. Bolted splice shall be provided with steel packing plates where necessary to ensure that the sum of any unintended steps between adjacent surfaces does not exceed 1 mm for HSFG bolted joints and 2 mm for other joints.

513.5.13.2. In welded butt joints, mis-alignment of parts to be joined shall not exceed the lesser of 0.15 times the thickness of thinner parts or 3 mm. However, if due either to different thicknesses arising from rolling tolerances or a combination of rolling tolerances with above

permitted mis-alignment, this deviation is more than 3 mm, it shall be smoothened by a slope not steeper than 1:4.

513.6. **Inspection and Testing**

513.6.1. **General :** No protective treatment shall be applied to the work until the appropriate inspection and testing has been carried out. The stage inspection shall be carried out for all operations so as to ensure the correctness of fabrication and good quality.

513.6.2. **Testing of Material**

513.6.2.1. Structural steel shall be tested for mechanical and chemical properties as per various IS as may be applicable and shall conform to requirements specified in IS:2062-1984, IS:11587-1986, IS:1977-1973, IS:8500-1977, IS:961-1975, etc.

513.6.2.2. Rivets, bolts, nuts, washers, welding consumables, steel forging, casting and stainless steel shall be tested for mechanical and chemical properties as applicable and shall conform to requirements, as specified in the appropriate Indian Standard.

513.6.3. Rolling and cutting tolerance shall be as per IS:1852. The thickness tolerance check measurements for the plates and rolled sections shall be taken at not less than 15 mm from edge.

513.6.4. Laminations in plates shall be carried out for areas specified in Clause 513.2 by ultrasonic testing or any other specified methods. Flame cut edges without visual signs of laminations need not be tested for compliance with Clause 513.2 unless specified otherwise by the Engineer.

513.6.5. Steelwork shall be inspected for surface defects and exposed edge laminations during fabrication and blast cleaning. Significant edge laminations found shall be reported to the Engineer for his decision.

Chipping, grinding, machining or ultrasonic testing shall be used to determine depth of imperfection.

For dynamically loaded structures recommended criteria for allowable discontinuities for edge defects and the repair procedure shall be as given in Table 13.2, until and unless specified otherwise. The weld procedure shall be as appropriate to the material.

Table 13.2. Discontinuity of Edge

	Discontinuity	Repairs required
1.	Discontinuities of 3 mm in maximum depth, any length for material thickness upto 200 mm	None
2.	Discontinuities of 3 mm to 6 mm in depth and over 25 mm in length for thickness upto 100 mm and 6 mm to 12 mm depth, over 50 mm in length for thickness 100 mm to 200 mm	Remove, need not be welded
3.	Discontinuities of 6 mm to 25 mm in depth, over 25 mm in length for thickness upto 100 mm and 12 mm to 25 mm in depth, over 25 mm in length for thickness over 100 to 200 mm	Remove and weld, No single repair shall exceed 20 per cent of edge being repaired
4.	Discontinuities over 25 mm in depth, any length for thickness 100 to 200 mm	With approval of Engineer remove to depth of 25 mm and repair by weld block
5.	On edges cut in fabrication, discontinuities of 12 mm maximum depth any length	None

513.6.6. **Measurement of curvature and camber:** Horizontal curvature and vertical camber shall not be measured for final acceptance before all welding and heating operations are completed and flanges have cooled to uniform temperature. Horizontal curvature shall be checked with girder in the vertical position by measuring offsets from a string line or wire attached to both flanges or by any other suitable means. Camber shall be checked by adequate means.

513.6.7. **Tolerance for drilled and reamed holes:** Acceptable deviation in holes drilled and reamed for mild steel and high strength rivets, bolts of normal accuracy and also for high strength friction grip bolts should be as per appropriate Indian Standard.

513.6.8. **Bolted connections**

513.6.8.1. Bolted connection joints with black bolts and high strength bolts shall be inspected for compliance of requirements mentioned in Clause 513.5.5.

The Engineer shall observe the installation and tightening of bolts so as correct tightening procedure is used and shall determine that all bolts are tightened. Regardless of tightening method used, tightening of bolts in a joint should commence at the most rigidly fixed or stiffest point and progress towards the free edges, both in initial snuggling and in final tightening.

The tightness of bolts in connection shall be checked by inspection wrench, which can be torque wrench, power wrench or calibrated wrench.

Tightness of 10 per cent bolts, but not less than two bolts, selected at random in each connection shall be checked by applying inspection torque. If no nut or bolt head is turned by this application connection can be accepted as properly tightened, but if any nut or head has turned all bolts shall be checked and if necessary retightened.

513.6.8.2. Bolts, and bolted connection joints with high strength friction grip bolts shall be inspected and tested according to IS:4000-1967.

513.6.9. Rivets and riveted connection shall be inspected and tested for compliance or requirements mentioned in Clause 513.5.6.

The firmness of the joint shall be checked by 0.2 mm filler gauge, which shall not go inside under the rivet head by more than 3 mm. There shall not be any gap between members to be riveted.

Driven rivets shall be checked with rivet testing hammer. When struck sharply on head with rivet testing hammer, rivet shall be free from movement and vibration.

All loose rivets and rivets with cracked, badly formed or deficient heads or with heads which are unduly eccentric with shanks, shall be cut out and replaced.

513.6.10. **Alignment of joints:** The alignment of plates at all bolted splice joint and welded butt joints shall be checked for compliance with requirements of Clause 513.5.13.

513.6.11. Testing of flame cut and sheared edges is to be done where the hardness criteria of Clause 513.5.4.2 (a) to (d) are adopted. Hardness testing shall be carried out on six specimens.

513.6.12. **Welding and welding connection**

513.6.12.1. Welders qualification test shall be carried out as per requirements laid down in IS:7318 (Part- I), for respective approved welding procedure, they shall satisfy relevant requirements of IS:7310 (Part-I)-1970.

Welding procedure, welded connection and testing shall be in compliance of requirements mentioned in Clause 513.5.7.

513.6.12.2. All facilities necessary for stage inspection during welding and on completion shall be provided to the Engineer or their inspecting authority by manufacturer.

513.6.12.3. Adequate means of identification either by an identification mark or other record shall be provided to enable each weld to be traced to the welder(s) by whom it was carried out.

513.6.12.4. All metal arc welding shall be in compliance with IS:9595 provisions.

513.6.12.5. The method of inspection shall be according to IS:822-1970 and extent of inspection and testing shall be in accordance with the relevant standards or in the absence of such a standard, as agreed with the Engineer.

513.6.12.6. **Procedure tests:** The Destructive and Non-Destructive test of weld shall be carried out according to IS:7307 (Part I).

513.6.12.7. **Non-Destructive testing of welds:** One or more of following methods may be applied for inspection or testing of weld.

513.6.12.7.1. **Visual inspection:** All welds shall be visually inspected, which should cover all defects of weld, such as, size, porosity, crack in the weld or in the HAZ (Heat affected zone), etc. Suitable magnifying glass may be used for visual inspection. A weld shall be acceptable by visual inspection if it shows that :

(a) The weld has no crack.

(b) Through fusion exist between weld and base metal and between adjacent layers of weld metal.

(c) Weld profiles are in accordance with requisite clauses of IS:9595 or as agreed with the Engineer.

(d) The weld shall be of full cross-section, except for the ends of intermittent fillet welds out side of their effective length.

(e) When weld in transverse to the primary stress, under-cut shall not be more than 0.25 mm deep in the part that is under-cut and shall not be more than 0.8 mm deep when the weld is parallel to the primary stress in the part that is undercut.

(f) The fillet weld in any single continuous weld shall be permitted to underrun the nominal fillet weld size specified by 1.6 mm without correction provided that under-size portion of the weld does not exceed 10 per cent of the length of the weld. On the web-to-flange welds on girders, no under-run is permitted at the ends for a length equal to twice the width of the flange.

(g) The piping porosity in fillet welds shall not exceed one in each 100 mm of weld length and the maximum diameter shall not exceed 2.4 mm, except for fillet welds connecting stiffeners to web where the sum of diameters of piping porosity shall not exceed 9.5 mm in any 25 mm length of weld and shall not exceed 19 mm in any 300 mm length of weld.

(h) The full penetration groove weld in butt joints transverse to the direction of computed tensile stress shall have no piping porosity. For all other groove welds, the piping porosity shall not exceed one in 100 mm of length and the maximum diameter shall not exceed 2.4 mm.

513.6.12.7.2. **Magnetic particle and radiographic inspection:** Welds that are subject to radiographic or magnetic particle testing in addition to visual inspection shall have no crack.

Magnetic particle test shall be carried out for detection of crack and other discontinuity in the weld according to IS:5334.

Radiographic test shall be carried out for detection of internal flaws in the weld, such as, crack, piping porosity, inclusion, lack of fusion, incomplete penetration etc. This test may be carried out as per IS: 1182 and IS: 4853.

Acceptance criteria:

The weld shall be unacceptable if radiograph or magnetic particle testing shows any of the type discontinuities listed hereunder, unless agreed by the Engineer.

(a) For welds subjected to tensile stress, the greatest dimension of any porosity or fusion type discontinuity that is 1.6 mm or larger in greatest dimension shall not exceed the size, "B", for the effective throat or weld size. The distance from any porosity or fusion type discontinuity described above to another such discontinuity to an edge, or to the toe or root of any intersecting flange to web shall not be less than the minimum clearance allowed C, for the size of discontinuity under examination.

(b) For welds subjected to compressive stress only, the greatest dimension of porosity or a fusion type discontinuity that is 3.2 mm or larger in greatest dimension shall not exceed the size, B, nor shall the space between adjacent discontinuities be less than the minimum clearance allowed, C, for the size of discontinuity under examination.

(c) The discontinuities having greatest dimension of less than 1.6 mm shall be unacceptable, if the sum of their greatest dimension exceed 9.5 mm in any 25 mm length

of weld, over and above requirements mentioned in (a) and (b) above.

(d) The limitations for 38.1 mm effective throat shall be applicable to all effective throats greater than 38.1 mm thickness.

(e) For welded bridge girders structural steel to IS:2062 and IS: 8500 only shall be used except for secondary members, such as, bracings, etc.

513.6.12.7.3. **Ultrasonic inspection:** The ultrasonic testing in addition to visual inspection shall be carried out for detection of internal flaws in the weld, such as, cracks, piping porosity inclusion, lack of fusion, incomplete penetration, etc. Acceptance criteria as per IS:4260 or any other relevant IS Specification and as agreed by the Engineer.

513.6.12.7.4. **Liquid penetrant inspection:** The liquid penetrant test shall be carried out for detection of surface defect in the weld, as per IS:3658, in addition to visual inspection.

513.6.12.7.5. The non-destructive testing of following welds be carried out using one of the methods or methods described in Clause 513.6.12.7.2 to 4 as may be agreed by the Engineer.

(a) All transverse butt weld in tension flange.

(b) 10 per cent of length of longitudinal butt welds in tension flange

(c) 5 per cent of the length of longitudinal and transverse butt welds in compression flanges

(d) All transverse butt welds in webs adjacent to tension flanges as specified by the Engineer.

The particular length of welds to be tested shall be agreed with the Engineer, in case of (b) and (c).

Where specified by the Engineer, bearing stiffeners or bearing diaphragms adjacent to welds, flange plates adjacent to web/flange welds, plates at cruciform welds, plates in box girder construction adjacent to corner welds or other details shall be ultrasonically tested after fabrication.

Any lamination, lamellar tearing or other defect found shall be recorded and reported to the Engineer for his decision.

513.6.12.8. **Testing of welding for cast steel:** The testing of weld for cast steel shall be carried out as may be agreed by the Engineer.

513.6.12.9. **Stud shear connectors:** Stud shear connectors shall be subject to the following tests :-

(a) The fixing of studs after being welded in position shall be tested by striking the side of the head of the stud with 2 kg hammer, to the satisfaction of the Engineer.

(b) The selected stud head stroked with 6 kg hammer shall be capable of lateral displacement of approximately 0.25 height of the stud from its original position. The stud weld shall not show any signs of cracks or lack of fusion.

The studs whose welds have failed the tests given in (a) and (b) shall be replaced.

513.6.12.10. **Inspection of members and components**

513.6.12.10.1. **Inspection requirement:** The fabricated member/ component made out of rolled and built-up section shall be checked for compliance of the tolerances given in Table 13.1. Inspection of member/components for compliance with tolerances, the check for deviations shall be made over the full length.

During checking the inspection requirement shall be placed in such a manner that local surface irregularities do not influence the results.

For plate, out-of-plane deviation shall be checked at right angle to the surface over the full area of plate.

The relative cross girder or cross frame deviation shall be checked over the middle third of length of cross girder or frame between each pair of webs and for cantilever at the end of member.

The web of rolled beam or channel section shall be checked for out-of-plane deviation in longitudinal direction equal to the depth of the section.

During inspection, the component/member shall not have any load or external restraint.

513.6.12.10.2. **Inspection stages:** The inspection to be carried out for compliance of tolerances shall include but not be limited to the following stages:

(a) For completed parts, component/members on completion of fabrication and before any subsequent operation, such as, surface preparation, painting transportation, erection.

(b) For webs of plate and box girder, longitudinal compression flange stiffeners in box girders, and orthotropic decks and all web stiffeners at site joints, on completion of site joint.

(c) For cross girders and frames, cantilevers in orthotropic decks and other parts in which deviations have apparently increased on completion of site assembly.

513.6.12.10.3. Where, on checking member/component for the deviations in respect of out-of-plane or out-of-straightness at right angles to the plate surface, and any other instances, exceed tolerance, the maximum deviation shall be measured and recorded. The recorded measurements shall be submitted to the Engineer, who will determine whether the component/member may be accepted without rectification, with rectification, or rejected.

514. HANDLING, TRANSPORTATION AND ERECTION

514.1. Scope

514.1.1. This clause lays down guidelines of general nature for handling, transportation and erection of bridges and their components.

514.1.2. It deals with the action to be taken for various operations in handling, transportation in shop floor and in transit as also in the erection site.

514.2. Transportation and Handling

514.2.1. The Engineer should plan the transportation and mention the mode of transportation, packing, placement, fastening of components or materials to ensure carriage free from damages or undue distortion. When deciding the mode, the route should be surveyed and local restriction in terms DO's/DON'T's statement for proper handling/transportation to be issued.

514.2.2. All transportable consignments should carry dispatch advice/challan as per directions to party concerned. Depending on "Target Factor" requirement of materials to be adjusted.

514.2.3. Loose assembled or sub-assembled items should have clear match mark number of the erection drawing. Critical items should be taken special care.

514.2.4. Protruded members to be specially protected during transit. Threaded and machined portion of fabricated structures should be carefully handled against damage.

514.2.5. Small items, e.g., nuts, bolts, washers, packing plates rivets electrodes shall be dispatched in containers and details fully listed to ensure proper receipt and storage. Under loaded consignments should be normally avoided.

514.2.6 In case of heavy and unusual structures, availability of the transportation medium should be checked in advance and arrangements tied up. Stability of the members shall be checked during loading or transportation. Necessary safety measures shall be ensured.

514.2.7. For access to the erection site, it may be necessary to erect temporary road bridges which can allow safe movement of the fabricated materials and equipment.

514.3. **Storage**

514.3.1. Suitable area for storage of structures and components shall be located near the site of work. The access road should be free from water logging during the working period and the storage area should be on a leveled and firm ground.

514.3.2. The store should be provided with adequate handling equipment, e.g., road mobile crane, gantries, derricks, chain-pulley blocks, winch of capacity as required. Stacking area should be planned and have racks, sleeper stands, access tracks and properly lighted.

514.3.3. Storage should be planned to suit erection work sequence and avoid damage or distortion.

514.3.4. Fabricated materials are to be stored on non-corroding surfaces with erection marks visible, such as, not to come in contact with earth surface or water and should be accessible to handling equipment.

514.3.5. Small fittings, hand tools, are to be kept in containers in covered stores.

514.4. **Handling**

514.4.1. IS:7293 and IS:7969 dealing with handling of materials and equipment for safe working should be followed. Safety nuts and bolts as directed are to be used while working.

514.5. **Erection Scheme**

514.5.1. Design of a bridge should take into consideration the method of erection. A detailed scheme must be prepared showing stage-wise activities, with complete drawings and working phase-wise instructions. This should be based on detailed stage-wise calculation and take into account specifications and capacity of erection equipment machinery, tools, tackles to be used and temporary working loads as per codal provisions.

514.5.2. The scheme should be based on site conditions, e.g., hydrology rainfall, flood timings and intensity, soil and subsoil conditions in the riverbed and banks, max. water depth, temperature and climatic conditions, available working space, etc.

514.5.3. The scheme should indicate detail of materials required with specifications and quantities, type of storage required, etc.

514.5.4. The scheme should indicate precisely the type of temporary fasteners to be used as also the min. percentage of permanent fasteners to be fitted during the stage erection. The working drawing should give clearly the temporary jigs. fixtures, clamps, spacers supports, etc. Adequate provision of spares of vulnerable items to be made.

514.6. **Procedure of Erection**

514.6.1. Prior to actual commencement of erection, all equipment, machinery, tools, tackles, ropes, etc. need to be tested to ensure their efficient working. Frequent visual inspection is essential in vulnerable areas to detect displacements, distress, damages, etc.

514.6.2. Deflection and vibratory tests shall be conducted in respect of supporting structures, launching truss as also the structure under erection and unusual observations reviewed. Looseness of fittings are to be noted.

514.6.3. For welded structures, welders' qualification and skill are to be checked as per standard norms. Non-destructive tests of joints as per designer's directives are to be carried out.

514.6.4. Precision non-destructive testing instruments available in the market should be used for noting various important parameters of the structures frequently and systematic record is to be kept.

514.6.5. Safety requirements should conform to IS:7205, 7273 & 7269 as applicable and should be a consideration of safety economy and rapidity.

514.6.6. Erection work should start with complete resources mobilized as per latest approved drawings and after a thorough survey of foundations and other related structural work. In case of work of magnitude, maximum mechanization is to be adopted.

514.6.7. The structure should be divided into modules, as per the scheme. This should be pre-assembled in a suitable yard/platform and its matching with members of the adjacent module checked by trial assembly before erection. Such assembled girders may be tested with simulated loads in case of erection on difficult terrain.

514.6.8. The structure shall be set out to the required lines and levels. The strokes and masses are to be carefully preserved. The steelwork should be erected, adjusted and completed in the required position to the specified line and levels with sufficient drifts and bolts.

Packing materials are to be available to maintain this condition. Organized "Quality Surveillance" checks need to be exercised frequently.

514.6.9. The method of erection, the drawing of temporary work and the use of erection equipment shall be subject to the approval by the Engineer.

514.6.10. **Joints:** Any connection to be riveted or bolted shall be secured in close contact by service bolts or specified no. of permanent bolts before final connection. Service bolts are to be fully tightened and kept as such by torque wrenches when the joint is assembled. Joints shall be made by filling not less than 50 per cent of the holes with service bolts and drifts in the ratio of 4:1. Connections to be completed by close tolerance bolts or as specified, as such, practicable.

514.6.11. Any connection to be site welded shall be securely held in position by approved means to ensure accurate alignment, camber and position before welding is commenced.

Appendix-A

LIMITATIONS

(Clause 502)

A1. The following special steel bridge structures have not been covered in the present code :

(a) *Curved Bridges*

For curved bridges, rigorous analysis should be made and detailing must follow the needs of the curvature effects. Having taken into consideration the above aspects, provisions of this code can be applied to curved bridges as appropriate.

(b) *Cable Stayed Bridges* and

(c) *Suspension Bridges*

These are special types of bridges calling for specialised treatment both for analysis and design. Also, erection conditions need to be thoroughly analysed.

(d) *Temporary* and

(e) *Pedestrian Bridges*

Because of their nature of use, certain provisions of the code (such as, permissible deflection, live load, etc.) can be relaxed subject to the approval of the Engineer

(f) *Swing Bridges and*

(g) *Bascue Bridges*

These type of bridges involve mechanical equipment for which relevant codes need to be referred. For structural portion of analysis and design, provisions of the code as appropriate can be applied.

(h) *Box Girder Bridges*

(i) *Prestressed Steel Bridges*

A2. Also, the following aspects of steel road bridges have not been covered in this code :

(a) *Ratings of Bridges*

For this aspect, IRC:SP:37-1991 "Guidelines for Evaluation for Load Carrying Capacity of Bridges" may be followed.

(b) *Fatigue*

Fluctuation of stresses may cause fatigue failure for members or sections at lower stresses than those at which they would fail under the load. However, for steel road bridges, this aspect is not always critical and has, therefore, not been included in this Code. Members subjected to fluctuation of stresses need to be examined in accordance with IS:1024-1979 (Ref. Clause 505.5).

Appexdix-B

RULES FOR CAMBERING PRE-DEFORMED OPEN WEB GIRDER SPANS

[Clauses 506.8.3.1 & 507.6.2 (b)]

B1. **Preparation of Camber Diagram**

B1.1. Contract drawings are dimensioned for the main girder without camber and in order to ensure that its fabrication and erection shall be, such as, to eliminate deformation stresses in the loaded span, a camber diagram shall be prepared on which shall be clearly indicated the amounts by which the nominal lengths (i.e.) the lengths which will not give camber) of members shall be increased or decreased in order that the outline of the girder under full load (dead load and 75 per cent live load without impact), shall be the nominal outline. A further change as indicated in para 1.4 may be made when the outline of the girder shall be normal outline, enlarged (1 + K) times in the case of a through span and reduced (1 - K) times in the case of a deck span (see para 1.4 below for definition of K).

B1.2. The stress camber change in each member shall be equal to the change of length of member due to the above loading, but of opposite sign.

B1.3. For the purpose of calculating the change in length of members under stress, the modulus of elasticity for both high tensile and mild steel shall be taken as 2.11×10^5 MPa. The effective length shall be taken between the theoretical intersection points of adjacent members.

B1.4. To ensure that the length of the floor system of a span shall be constructed to its nominal dimensions, i.e., to avoid changes in lengths of floor and loaded chord lateral system

a further change in length shall be made in the lengths of all members equal to :

$$\frac{\text{Loaded chord extension or contraction}}{\text{Loaded chord length}} \times \text{length of member} = K \times L$$

In through spans this change will be an increase in the lengths of all members while in the case of deck spans it will be a decrease in the lengths of all members.

B1.5. The nominal girder lengths altered in accordance with paragraphs 1.1 and 1.4 give a girder correctly stress cambered but with the loaded chord length identical with that shown on the contract drawings, thus requiring no modifications to floor and loaded chord lateral systems.

B1.6. The nominal lengths and camber lengths shall be rounded off to the nearest half a millimeter.

B1.7. The difference between nominal lengths and camber lengths thus, modified is the practical camber changes.

B1.8. The ordinates corresponding to the required camber at nodes may be obtained either by drawing a Williot Mohr Diagram or any other acceptable method.

B1.9. Adjustments of the lengths shall be made to top lateral bracing members to suit camber lengths of the top chords in the case of through girder spans and to the bottom lateral bracing members in the case of deck spans. The average value of the pre-stressed length of top or bottom lateral member, as the case may be. shall be adopted throughout.

B2. **Fabrication**

B2.1. The actual manufactured lengths of the members are to be the lengths "with camber" given on the camber diagram.

B2.2. The positions and angular setting out lines of all connection holes in the main gussets and also the positions of the connection holes in the chord joints and the machining of

the ends shall be exactly as shown on the contract drawings. This will permit the butts in the chord segments to be exactly as shown on the contract drawings.

B2.3. The groups of connection holes at the ends of all the members are to be as shown on the contract drawings, i.e., without any allowance for camber but the distance between the groups at the ends of each member shall be altered by the amount of the camber allowance in the member.

B3. **Erection**

B3.1. The joints of the chords shall be drifted, bolted and preferably riveted to their geometric outline.

B3.2. All other members are to be elastically strained into position by external forces, so that as many holes as possible are fair when filled with rivets.

B3.3. Drifting of joints shall be avoided as far as possible, and when necessary, should be done with great care and under close expert supervision. Hammers not exceeding one kg. in weight should be used with turned barrel drifts and a number of holes drifted simultaneously, the effect of the drifting being checked by observation of adjacent unfilled holes.

B3.4. The first procedure during erection consists of placing camber jacks in position on which to support the structure. The camber jacks should be set with their tops level and with sufficient run out to allow for lowering of panel points except the centre by the necessary amounts to produce the required camber in the main girders. It is essential that the camber is accurately maintained throughout the process of erection and it should be constantly checked. The jacks shall be spaced so that they will support the ends of the main girders and the panel points. The bottom chord members shall then be placed on the camber jacks, carefully

leveled and checked for straightness and the joints made and riveted up.

B3.5. The vertical and diagonal web members, except the posts, shall then be erected in their proper positions on the bottom chords. It is recommended that temporary top gussets, the positions of the holes in which are corrected for the camber change of length in the members, should be used to connect the top ends of the members; this will ensure that the angles between the members at the bottom joints are as given by the nominal outline of the girders. The vertical and diagonal shall then be riveted to the lower chords.

B3.6. All panel points, except the centre, shall now be lowered by amounts to produce the correct camber in the main girders as shown on the camber diagram.

B3.7. The top chord should be erected piece by piece working symmetrically from the centre outwards, and the joint made by straining the members meeting at the joint and bringing the holes into correct registration.

B3.8. The temporary gussets, if used, shall be replaced by the permanent gussets in the same sequence as the erection of the top boom members.

B3.9. The end posts shall be erected last. The upper end connection should preferably be made first and if there is no splice in the end raker, the final closure made at the bottom end connection. If there is a splice, the final closure should be made at the splice.

B3.10. When cantilever method of erection is used, the above procedure does not apply.

Appendix-C

PROTECTION AGAINST CORROSION

C1. Introduction

C1.1. Phenomenon of corrosion is essentially recognized as an eletro-chemical process through formation of anode (steel substrata), cathode (top mill scale) in the all pervading presence of electrolyte (moisture).

C1.2. Concentration of oxygen, carbon dioxide, chlorides causing acid film and unfavourable temperature, residual stresses, etc. are some of the factors which initiate and accelerate corrosion. An everlasting solution to the problem of protection against corrosion is yet to be evolved.

C1.3. All steel work should be designed and detailed to minimise the risk of corrosion. Provision of IS:8629 should be taken into account for corrosion protection.

All parts should be accessible for inspection, cleaning and painting or should be effectively sealed against corrosion. Where these methods are not possible either the surface of the steel should be given a system of protective coating or treatment selected with due regard to the design life of the part, together with an additional thickness of the steel or the steel used should have corrosion resistant properties as suitable for the design environment.

Adequate drainage should be provided where there are chances of collection of water. Hollow members without access for internal inspection and maintenance should be effectively sealed against corrosion.

C2. Aspects to be considered

C2.1. Prior to choosing and specifying the type of protective system for bridge structures, it is necessary to:

(a) Identify the environment

(b) Assess life cycle costs for some prima-facie suitable systems

(c) Compare and select the preferred system

(d) Define the system as completely as possible

Selection of a protective system and the correct specifications of material including testing and their application methods are essential and important for satisfactory performance of the chosen system under given environmental conditions

C2.2. *Environment*

Following aspects need to be studied :-

C2.2.1. *Broad environmental conditions :*

(a) Exterior exposed

(b) Polluted/non polluted (Inland)

(c) Polluted/non polluted (Coastal)

(d) Exterior sheltered

(e) Saline/non-saline

C2.2.2. *Local conditions :*

(a) Splash/Fully immersed zone;

(b) Presence of harmful salts, viz. sulphates/chlorides

(c) Likelihood of abrasion and impact

(d) Presence of fungi and bacteria

C2.2.3. *Other factors :*

(a) Appearance (after painting) vis-à-vis the surroundings

(b) Maintenance aspects

- Access for effective inspection and maintenance
- Possibility of effective maintenance.

- Time schedule for renewal of coating system

(c) Tolerance of coating

- In different surface preparation
- In different application treatments

(d) Suitability of coating system from consideration such as :

Ready availability, compatibility, application methods such as brushing airless spray, etc., time schedule between first coating and cost effectiveness final treatment, damage during transport and handling, etc.

(e) Past performance of the coating system

(f) Level of expertise required in application of coating system.

C3. Control of Corrosion

Corrosion can be minimised/controlled by :

- Improved design and detailing;
- Cathodic protection;
- Concrete encasement;
- Use of special corrosion resistant steels;
- Protective coating on surface.

Salient features concerning the above are given hereunder :-

C3.1. *Improved design and detailing*

This essentially will entail :

- Selection of appropriate quality of steel;
- Proper slope and geometry of structures to avoid accumulation of deleterious matter and debris;
- Easy accessibility to all parts of structures;
- Proper drainage hole locations; and draining out without affecting structural members.

- Provision of good seal in bolted connections; (to avoid corrosion in the crevices; by trapped moisture);
- Avoidance of sharp cut-edges;
- Ensuring Electro-chemical compatibility between the coating and parent-metals;
- Arrangements for safeguarding painted members from accidental damage during transport/erection).
- Extra thickness may be provided as per clause in the relevant chapter.

C3.2. *Cathodic Protection*

There are two basic types of cathodic protection :

(a) Sacrificial anode system: This system does not require external power source. Magnesium, Zinc, Aluminium are used as anode and the metal to be protected becomes cathode. Hence, there is no corrosion in the metal.

(b) Impressed current system: This system requires D.C. power supply.

C3.3. *Concrete Encasement*

The concrete itself acts as a protective barrier. Adequate cover and good quality of concrete is however, essential. Loose mill scale and rust should be removed but other surface preparations are not necessary. The steel work must not be painted but a cement-wash may be given to the steel surface to prevent corrosion until it is encase.

C3.4. *Use of Corrosion Resistant Steel*

C3.4.1. Weathering steels are low-alloyed steels containing a total of 1 per cent - 2 per cent alloys, in particular, copper, chromium, nickel and phosphorous. The chemical composition of this steel is such that increased protection is provided by the patina which develops after a short period of oxidation. Atmospheric that no painting is required for such steels is not justified as recent experience of such steels,

exposed to saline environment, (where chlorides ions act as de-passivators indicated that weathering steels act as de-passivators) indicates that weathering steels have also some corrosion problems. Fabrication and erection of such steels need greater care.

C3.4.2. *Stainless Steel*

Another well known non-corrosive type of steel is the stainless steel. Stainless steels are known for their resistance to atmospheric corrosion. However, such steels also tend to corrode particularly where they are protected from direct oxygen supply or where oxygen availability is reduced. Adverse environmental conditions, such as, warm moisture, saline charged atmosphere is also detrimental to long term satisfactory performance of these steels. Use of these may be considered only as special components.

C3.5. *Protective Surface Coating*

C3.5.1. All protective coatings are classified separately as

(a) Metallic; e.g. galvanizing, metallising, etc.

(b) Non-metallic, e.g., paints

C3.5.2. *Galvanising*

The process of providing zinc coating to steel surface is called galvanisation.

C3.5.3. *Metallising*

Metallising consists of projecting an atomised stream of molten metal at a high velocity from a special gun on to a 'prepared' surface.

C3.5.4. *Painting*

Paints are a mixture of film forming material called a vehicle or binder and pigments for colouring and protection. A drying agent may also be added. The vehicle or binder may be natural oil or resin or alkyd varnish or synthetic resin.

Pigments, are solid particles, insoluble in the binder and add colour to the paint or corrosion resistance.

C3.5.4.1. *Surface Preparation*

Steel surface to be painted either at the fabricating shop or at the site of work shall be prepared in a thorough manner with a view to ensuring complete removal of mill scale by one of the following processes as agreed to between the fabricator and the Engineer (or the purchaser).

(a) Grit/sand blasting

(b) Pickling which should be restricted to single plates, bars and sections

(c) Flame cleaning

(d) Scraping and wire brushing

The last process, viz., scraping and wire brushing should be avoided as far as possible and restricted to temporary bridges only.

Primary coat shall be applied as soon as practicable after cleaning and in case of flame cleaning, primary coat shall be applied while the metal is warm.

All slag from welds shall be removed before painting. Surfaces shall be maintained dry and free from dirt and oil. Work out of doors in frosty or humid weather shall be avoided.

C3.5.4.2. *Type of Paints*

C3.5.4.2.1. *Ordinary paints*

These include paints based on drying oils, alkyd resin, modified alkyd resin, phenolic varnish epoxy, etc.

Alkyd resin paints for the protection of steel structures are based partly on natural oils and partly on synthetic resins. These paints are used for steel structures in atmospheres which are not too aggressive.

Oil based paints are only used for steel structures in cases where the surface preparation cannot be ideal. Ordinary painting system can generally be sub-divided into two groups.

(a) *Primary Coats*

This is applied immediately after the surface preparation and should have the properties of

- Adhesion
- Corrosion inhibition and
- Imperviousness to water and air.

(b) *Finishing Coats*

These are applied over the primary coats and should have the properties of

- Durability
- Abrasion resistance, and
- Aesthetic appearance and smooth finish.

Depending on the aggressive nature of the environment the number of coats can be increased, e.g., A primer, under-coat and top-coat can be given for aggressive marine atmosphere.

C3.5.4.2.2. *Chemical resistant paints*

The more highly corrosion resistant paints can be divided in two main groups

- One pack paints (ready for use)
- Two pack paints (mixed before use).

The two pack paints are mixed together immediately before use and are workable thereafter only during a restricted period of time. They dry as a result of a reaction between their components and yield hard tough films with resistance to abrasion.

C3.5.4.2.3. *Vinyl paints*

Vinyl paints are based on polyvinyl resins, such as, polyvinyl-chloride (PVC) and polyvinyl-acetate, etc.

Certain types of vinyl resin paints yield thick, relatively soft and rubber, like, coatings with good chemical resistance. They can be repainted without difficulty.

C3.5.4.2.4. *Chlorinated rubber paints*

These paints have also good chemical resistance. The main fields of applications are in aggressive environments and in the chemical industry. In general, chlorinated rubber paints do not have a high gloss.

C3.5.4.2.5. *Bituminous paints*

As a paint vehicle, bitumen is inferior to other vehicles but because of the low price, this could be applied in greater thickness (up to several millimeters) and may be suitable for some situation. A significant advantage of bitumen paints is their impermeability to ingress of water. However, bituminous paints do not withstand effectively detrimental effects of oil.

C3.5.4.2.6. *Epoxy paints*

These resin paints have good adherence to a very well prepared surface. They are mechanically strong and resistant to chemicals. A disadvantage of epoxy resin paints is that it can rapidly become dull when exposed to strong sun light. These disadvantages do not, however, greatly influence their protective power.

C3.5.4.2.7. *Polyurethane paints :*

The chemical and mechanical behaviour of polyurethane paint resemble very much those to the epoxy paints. However, polyurethane paint retains its gloss for a longer period.

C3.5.4.2.8. *Zinc rich paints :*

Instead of introducing an inhibitive pigment into paint, metallic zinc can be used, and such paints can provide cathodic protection to steel.

C3.5.4.3. *Choice of Painting System*

The choice of suitable painting system is dependent on factors such as :

- Available application methods viz. brush, roller or spray;
- Durability in a specific environment
- Availability of skilled manpower
- Cost/benefit, etc.

It is therefore necessary to consult various manufacturer of paint and ascertain the above aspects while deciding on the appropriate choice of painting system.

C3.5.4.4. Typical guidelines for epoxy based paints and the conventional painting system for bridge girders are given below :

(a) *Epoxy Based Painting*

(i) Surface preparation - Remove oil/grease by use of petroleum hydrocarbon solution (ISI :1745-1978). Grit blasting to near white metal surface.

(ii) Paint system - 2 coats of epoxy zinc phosphate primer = 60 micron.

Total 5 coats = 200 micron.

(b) *Conventional Painting System*

(i) (For areas where corrosion is not severe)

Priming coat:

One heavy coat of ready mixed paint, red lead primer to IS:102.

or

One coat of ready mixed paint zinc chrome primer to IS: 104 followed by one coat of ready mixed paint red oxide chrome primer to IS: 2074.

or

Two coats of zinc chromate red oxide primer to IS:2074

Finishing Coats:

Two cover coats of red oxide paint to IS: 123 or of any other approved paint shall be applied over the primer coat. One coat shall be applied before the fabricated steel work leaves the shop. After the steel work is erected at site, the second coat shall be given after touching up the primer and the cover coats, if damaged in transit.

(ii) (For areas where corrosion is severe)

Priming Coat

Two coats of ready mixed paint red lead primer to IS:102.

or

One coat of ready mixed zinc chrome primer to IS:104 followed by one coat of zinc chromate oxide primer to IS:2074.

Finishing Coats

Two coats of aluminium paint to IS:2339 shall be applied over the primer coats. One coat shall be applied before the fabricated steel work leaves the shop. After the steel work is erected at site the second coat shall be given after touching up the primer and the cover coat if damaged in transit.

C3.5.4.5. *General*

Surface which are inaccessible for cleaning and painting after fabrication shall be painted as specified before being assembled for riveting.

All rivets, bolts, nuts, washers, etc. are to be thoroughly cleaned and dipped into boiling linseed oil to IS:77.

All machined surfaces are to be well coated with a mixture of white lead to IS:34 and Mutton tallow to IS:887.

For site paintings the whole of the steel work shall be given the second cover coat after final passing and after touching up the primer and cover coats, if damaged in transit.

C3.5.4.6. *Quality of Paint*

The paints which have been tested for the following qualities as per the specification given in the relevant IS codes should only be used.

- Weight Test (weight per 10 lit. of paint thoroughly mixed);
- Drying times;
- Flexibility and adhesion;
- Consistency;
- Dry thickness and rate of consumption

C4. Guidelines for Protective Coating System in Different Environments

Corrosion has to be controlled in an economical way. Since the seriousness of the problem depends on atmospheric conditions and these vary enormously, there is no single protective system or method of application that is suitable for every situation.

C4.1. However, as a guide, broad recommendations are given in Table C-1 for various types of coatings in various environmental conditions, based on past experience as well as trials conducted overseas (both in laboratories and also in the field). Approximate life to first maintenance is also indicated and can be used as a guide for deciding on the maintenance schedule.

Table C-1. Recommendations for Types of Protective Coatings

	System	Environment
1.	Wire brush to remove all loose rust and scale; 2 coats drying oil type primer, 1 undercoat alkyd type paint; 1 finishing coat alkyd type. Total dry film thickness = 150μm.	Suitable for mild conditions where appearance is of some importance and where regular maintenance is intended This system may deteriorate to marked extent if it is exposed to moderate aggressive atmospheric conditions for lengthy period.
2.	Wire brush to remove all loose and scales; 2 coats drying oil type primary, 2 undercoats micaceous iron oxide (MXO) pigmented phenolic modified drying oil : Total dry film thickness : 170μ m.	Similar to (i) but where the appearance is not very important, provides longer life in mild condition. Will provide up to 5 years life to first maintenance in polluted inland environment.
3.	Blast clean the surface; 2 coats of quick drying primer under-coat alkyd type paint; Total dry film thickness = 130 - 150μ m.	Compared with (i) this would provide a longer life in mild conditions and could be used in less mild situation, e.g. inland polluted, where maintenance could easily be carried out at regular intervals.
4.	Blast clean the surface; 2 coats of drying oil type primer, 1 under-coat micaceous iron oxide pigmented drying oil type paint; Total dry film thickness=169-190μm.	Suitable for general structural steelwork exposed to ordinary polluted inland environments where appearance is not of primary importance.

5.	Blast clean the surface; 2 coats of metallic lead pigmented chlorinated rubber primer, 1 under-coat of high build chlorinated rubber, 1 finishing coat of chlorinated rubber : Total dry film thickness=200μm.	Suitable for structures in reasonably aggressive conditions, e.g., near the coast. Will provide long-term protection than (iv) in non-coastal situations. Also, suitable for aggressive interior situations such as industrial areas.
6.	Blast clean the surface : 350 μm. - 450 thickness coal tar epoxy.	This system would be suitable for sea-water splash zones or for conditions of occurrence of frequent salt sprays.
7.	Pickle; hot dip galvanise (Zinc) Total thickness = 85μ m.	Suitable for steel work in reasonably mild conditions. A life of 15-20 years before first maintenance could be expected in many situations.
8.	Grit blast, hot dip galvanised (Zinc) Total thickness = 140μ m.	Provides a longer life than (vii) because of the thicker zinc coating.
9.	Grit blast, 1 coat of sprayed zinc/aluminium followed by suitable sealer, Total thickness = 150μ m.	Expected to provide long term protection approx. 15-20 years in aggressive atmosphere.

Appendix-D

POST-CONSTRUCTION INSPECTION AND PREVENTIVE MAINTENANCE GUIDELINES

D1. General

Bridge structures, permanently exposed to atmosphere are subjected to effect of adverse environmental conditions. Investment made in the structural facility can be protected by well programmed, monitored-inspection and maintenance schedule adopted for its designed life. Such systems developed ensures structural safety by recording the state of the structure periodically and providing feed back information to designers while actually identifying actual and potential sources of trouble and taking remedial measures in time.

This clause lay down the desired inspection procedure for determining physical condition and programming maintenance needs of the bridges. Systematic periodical inspection required for various elements and the responsibilities of the inspection group have been specified. The scope of maintenance work involved does not include correcting measures for known deviations introduced during construction stage and no attempt has been made in this direction. It is necessary to understand that for proper inspection of various components of the bridge structure in built facilities should be developed at the detailing and construction stage, for accessibility to important areas.

D2. Inspection

Bridge inspection is done by use of well tried and established techniques required for assessing the physical condition of the structure. IRC Special Publication 35 : "Guidelines for Inspection and Maintenance of Bridges" may be referred in this connection.

D2.1. *Personnel*

D2.1.1. The in-charge for bridge inspection and reporting shall posses the following qualifications :

(a) Be a qualified engineer or equivalent with adequate experience in Bridge Inspection

or

(b) Have a minimum of 10 years experience in bridge inspection assignments in a responsible capacity.

D2.1.2. He shall be responsible for a methodical and thorough field inspection, the detailed analysis of all observation recorded, arrive at findings to recommend rectification of defects, imposition of speed restriction or load limitations and any other measures as necessary.

D2.1.3. The problems encountered in this work are variable and complex as such matured judgment is often required for evaluation of the recordings.

D2.1.4. He must be thoroughly familiar with design and construction features of the bridge so as to make a correct interpretation and capable of determining the safe load carrying capacity of the existing structure. He should be capable of recognising any deficiency in the structure, assess the seriousness and suggest appropriate remedial measures to ensure safety. His experienced knowledge to recognise problem areas (actual and potential) and to ensure preventive maintenance is an important requisite.

D2.1.5. He should be able to utilise the expert knowledge and skills of associate Engineers in respect of structural design, construction methods, material, hydro dynamics, equipment, soil technology, maintenance methods for permanent and emergency measures, etc. and should have access to resources and expert systems.

D2.1.6. Definite guidelines should be given to ensure availability of technical assistance from other agencies/where regular staff is not available. In case of specialised structure consultation with expert bodies is a necessity.

D3. **Training**

Bridge management requires extensive team work involving various levels of responsibility and skills. Training programmes need be framed in order to develop expertise. Training facilities may be set up at central level for training of trainees and at local level for actual imparting training to field staff, workshop on topical interest may be held regularly to acquaint the concerned people with accepted technical methods and their correct application.

D4. **Frequency of Inspection**

D4.1. *Detailed Inspection*

D4.1.1. The details and frequency to which Bridges are to be inspected will depend on such factors as age, traffic characteristics, state of the structure, vulnerability and having known history of deteriorating condition. Evaluation of these factors will be the responsibility of the individual in-charge of the inspection programme.

D4.1.2. Each bridge has to be inspected in detail at regular intervals not exceeding 5 years.

D4.2. *Periodic Routine Inspection*

Certain items in each bridge have to be inspected at definite intervals of time atleast once a year, irrespective of whether anything alarming has taken place or not.

D4.3. *Special Inspection*

Such Inspection are required for any bridge with known deficiencies, like, restriction on weight/speed, loss of

camber and are considered necessary on the basis of routine inspection or unusual occurrence.

Recommended frequencies of various inspection item are shown in Table D-1. These are for guidance only.

Table D-1. Bridge Inspection Record Sheet

Sl. No.	Inspection Items		5 Year	1 Year	6 Months
1.	Main Bridge	a) Steel Girders	*	*	
	Structure	and Stringers	*	*	
		b) Trusses			
2.	Bearing	a) Wearing surface	*	*	
3.	Decks	a) Drainage	*	*	As
		System	*	*	situation
		b) Steel Deck	*	*	demands
		c) Curbs	*	*	
		d) Foot Path	*	*	
4.	Expansion Joints		*	*	
5.	Railing and Crash Barriers		*	*	
6.	Signs		*		
7.	Services and Utilities		*		

Note : In the case of distressed bridge, special instructions to be issued by the competent authority.

D5. Inspection Procedure

D5.1. *General*

D5.1.1. The field inspection of a bridge should be conducted in a systematic and organised manner and observation recorded

on a format to ensure that no item is overlooked. Notes must be clear and detailed to the extent that they can be interpreted at a later date when report is prepared. Sketches and photographs should be included in an effort to record actual conditions.

D5.1.2. As far as possible the inspecting officer should schedule bridge inspection in those periods of the year which offer the most favourable conditions. Inspections during temperature extremes should be made at bearing, joints etc. Inspection should not be confined only to search for defects which exist, but for conditions of anticipatory nature and marking those zones. Preventive maintenance is equally important to corrective ones.

D5.2. *Inspection Items*

Inspection of all items, such as, approaches, waterways, basic floor conditions, substructure which may affect the safety of the steel superstructure need be done, like, other types of bridges.

D6. **Main Structure**

D6.1. Steel Girders and Stringers in the deck structure should be examined for signs of corrosion, cracks along the flanges around rivet or bolt heads, its contact surfaces and where water enters and stands or debris may collect at the ends.

D6.2. Flanges and webs shall be checked for any damage or misalignment. Web-stiffeners are to be examined for signs of deformation due to buckling. Unusual vibration or excessive deflection under passage of heavy loads should be noted and cause investigated.

D6.3. All end connections should be inspected to make sure than they are secure.

D6.4. Weld areas should be inspected to check crack. Special care should be exercised to inspect corners, curved, sections and

areas where there is an abrupt change in the size of metal or in configuration which may produce an area of concentrated stress or in areas where vibration or movement could produce stress concentration. Damages or deformation caused to the main-structural members due to vehicular impact should be particularly watched. Fatigue failure and welded joints being a cause of concern in bridge structures with age such structures need more careful check and watch.

D6.5. Creep - The longitudinal movement of a girder is termed as creep. This point should be checked and girders pulled back if necessary to the proper position.

D6.6. Distortion - With variation of temperature, the girder is likely to have longitudinal movement due to expansion and contraction, absolute freedom of movement is impracticable and there remains a residual force which develops internal stresses, causing tendency to distort. Distortion is measured in the bottom top chord.

D6.7. *Lateral Bracing*

Normally a span of a bridge consists of two or more girders braced together with lateral bracings. These bracings should be thoroughly checked for corrosion on loose rivets and deformity, viz., bracing distortion, etc.

D6.8. *Loose Rivets*

Field rivets are to be examined for looseness. This is caused due to running traffic and consequent vibratory effect and corrosion around the rivets. A joint with loose rivets should not be touched unless more than 20 per cent rivets in the joint are loose.

D6.9. *Bearings*

D6.9.1. All bearing devices should be examined to ascertain that

they are functioning properly. Changes in other parts of the structure, such as, pier/abutments settlement and tilt may be reflected in the bearing. Bearings should be seated properly on their bed plates and provided with gaps at both ends. Bearing assembly should be checked for possible cracks by magnifying glass after removal of paint cover in doubtful cases. In case of roller bearing, the relative position of the top castings, bottom castings and rollers variation to the temperature noted. Longitudinal movement of the free-end shall be recorded under moving load. After unusual occurrences bearings and support pads must be examined for cracks, etc. Lateral shear keys in skew bridges also invite special check.

D6.9.2. *Lubrication of Bearings*

Oiling and greasing of the bearing is done periodically once in 3 years. Improper and failure in timely lubrication may lead to corrosion of bearings resulting in reduction in strength and consequent damages.

Where bearings are encased in oil baths and stay submerged in recommended brand of lubricating oil the level of oil should be maintained on checking every year. It should be ensured that the oil baths are always sealed.

D6.9.3. *Elastomeric Bearings*

The physical conditions of elastomeric bearing pads should be inspected for observing any abnormal flattening. bulging or splitting which may indicate overloading or excessive unequal distribution of loading, shifting from original position should be checked particularly.

D6.9.4. *Condition of Bed-Block and H.D. Bolts*

Bed Blocks receive the full load from the bearings of the bridge and distribute and transmit the same to the masonry below. Restriction of free movement in superstructure may result in :

(1) Development of transverse cracks in piers/abutments

(2) Failure of bed blocks joints leading to shaking bed blocks

(3) Shearing of holding down bolts (Particular importance due to introduction of greater longitudinal forces)

It must be ensured that the anchor bolts are well secured.

D7. **Trusses**

D7.1. Camber of the trusses should be checked and the ambient temperature recorded at the time of detailed inspection. A camber diagram should be made in the inspection register. Loss of camber may be assessed from comparative readings.

D7.2. All truss members should be checked. The compression members should be checked for straightness absence of kink or bows and the connections are undisturbed. (Tension members should not show signs of cracking).

D7.3.1. The truss should be checked against damage due to collision with vehicular traffic, portal bracings and sway bracings are usually the most restrictive to overload movements and consequently susceptible to damage.

D7.3.2. The condition of pins at the connections and rivets, bolts should be checked to see that none are loose, worn-out or sheared. Particular care to be given to following locations :

(a) Connections of stringers to cross girders

(b) Connections cross girder to main girder

(c) End connection of bracings

(d) Chord joints and web-member connection

D8. **Corrosion and Painting**

Steel structure is sensitive to the atmospheric moisture and vehicular smoke and therefore should be protected by paints or anti-corrosive measures. The condition of the members

should be examined and the extent of corrosion recorded. The portions of steel work where water is likely to stangate or which are subjected to alternate wetting or drying need special care. Deformation in riveted or bolted multiple sections should be examined to check if moisture has entered and corroded the contact surfaces of the plates causing them to be pushed apart. The exact location and area of the affected portion should be recorded. This area should be got cleaned, thoroughly scraped, old paint, rust, scaling removed and repainted and appropriate remedial measures taken up immediately.

D9. **Decks**

D9.1. Steel decks should be checked for corrosion and unsound welds. It is important to maintain an impervious surface over a steel plate deck to protect against corrosion in aggressive environmental condition.

D9.2. It is necessary to have effective drain holes to prevent collection of water on the deck.

D10. **Expansion Joints**

D10.1. Maintenance of these joints need special attention and should be carefully examined. The joint should be clear of debris and be able to have free thermal expansion as designed.

D10.2. Finger type joints and sliding plate joints should be checked for loose anchorage, cracking or separation of welds or other defects. Such defects cause structural deformation and is hazardous to traffic. Deck adjacent to expansion joints should be carefully examined for voids and cracks. Underside of expansion joints also need careful inspection. Systematic documentation of the movement of expansion joints need to be kept to judge proper functioning of the bridge structure.

D11. **Railings and Safety Barriers**

D11.1. Handrails are to be examined for unusual damage, deformation, corrosion and paintings. The embedment of posts to be checked for rust stains, which are signs of rusting. Extent of corrosion need to be checked when signs exist on the surface.

D11.2. All handrails are to be checked for any damage for traffic. The vertical and horizontal alignment are to be maintained.

D12. **Services**

The number and types of utilities, such as pipelines, cables, etc. must be inspected and observations kept for record with the details suitably displayed. Special care need be kept for hazardous utilities, regular joint inspection in such case with suitable guidelines is a necessity.

D13. **Special Structures**

D13.1. *Moveable Bridges*

The most common type of moveable bridge are the swing span, vertical lift bascule (Single or double leaf). Inspection of the trusses, floor system, and other structural elements will require inspection procedures suitably modified as per guidelines mentioned in the Code. Ensuring proper seating of the girder after operation is an absolute necessity.

D13.2. In case of other structures, like, suspension bridges, cable stayed bridges detailed inspection manuals should be prepared and staff trained to observe the same.

D14. **Documentation**

D14.1. The most important function of bridge maintenance unit is to prepare a complete, methodical and current record for each bridge on the system. Much of the usefulness of the information obtained from field investigation depend upon its reliability and availability on a concise format. The record must be preserved systematically and readily available.

D14.2. Records should provide a full history of the structure

including all recommendations for strengthening and restoration works undertaken and the behaviour of the structure thereafter. This record should indicate clearly the life carrying capacity of the structure with supporting document calculations.

D14.3. Complete record in an usable format is vital for the continued service ability of the bridge. It is essential computerised data system is introduced as soon as practicable.

D14.4. A sample record sheet is shown in Table D-1.

D15. **Standard Tools**

A list of standard tools required for inspection is given in Table D-2 as guide.

Table D-2. Standard Tools and Equipment

A. STANDARD TOOLS

(1) Clip Board, Chalk, Markers, Clamps, etc.

(2) Pocket Tapes, Folding Rules, Tapes (10 m to 50 m) Feeler Gauges, Callipers, Micrometer Gauges,

(3) Straight Edge, Plumb Bob, Protector, Spirit Level.

(4) Thermometer, Inspection Mirror, Binoculars, Magnifying Glass, Camera.

(5) Scrappers, Emergy Paper, Portable Torque Wrenches, Light hammer, Piano-wire, Portable Ladder, Rope.

(6) Flash Light. Pocket Knife, Wire Brush, Chipping Hammer, Thin steel rod (for use as proper) (8 to 20 mm dia)

(7) Hydraulic Jacks, Pulley Blocks, Wire-Ropes, Chains, Slings, etc. of Ppropriate Capacity.

(8) Safety Equipment for Inpsecting Staff.

B. ASSESSMENT POSSIBLE

Steel	
Cracks	Ultrasonic, radiographic
Cable/Wire failure	Electric half cell potential
Corrosion meter	Electrical resistivity
Global behaviour	
Movements Instruments dial gauges	Modern Surveying
Extensometric measurements	Strain Gauges, and extensometer
Pressures, Forces	Pressures transducers, or load cells
Miscellaneous	
	Thickness of coating Paint film gauge (digital electrometer)
Water proofing membranes	Electric resistance
Vibration	Accelerometer
Widening of Cracks	Glass cell tabs
Metal thickness	Ultrasonic metal thickness measuring

D16. **Signs**

D16.1. All signs required to indicate restrictive load limit, reduced speed or impaired clearance should be inspected to ascertain they are visible and located in proper places. This inspection is to include sign at or on the structure and any advance warning sign. Examination should include that indications are legible and sign posts are well secured.

D16.2. For bridges over navigable, channels, it is necessary to inspect if the navigational signs for water-traffic are in place and secured. Navigational lights and serial obstructional lights should be inspected often to ensure that these are operating efficiently.
